Engineering News

VOLUME 74 NOVEMBER 11, 1915 NUMBER 20

Ornamental Concrete Elevated Railway, New York City

By Maurice E. Griest*

One of the new rapid-transit lines now being built in New York City passes along the Queens Boulevard, a 200-ft. automobile thoroughfare, for more than three-fourths of a mile. To avoid disfiguring the boulevard—for which a large expenditure had already been made—with a steel elevated railway of the ordinary type, careful study was given to working out a practical type of elevated-railway construction which should be ornamental, and less noisy than a plain open-floor steel structure; and at the same time cheaper than subway construction. The final selection was a new form of structure, a reinforced-concrete arcade or series of vaulted concrete arches.

An ornamental steel structure with columns curving into the longitudinal and cross-girders by sweeping curves was considered. The details were unsatisfactory, however, and no appreciable reduction in noise could be expected. The estimated cost, with open tie floor, was about 1.2 times the cost of a plain elevated structure.

By making the steelwork of plain form and detail, securing the flowing lines by means of a concrete covering or veneer (Fig. 2), fairly satisfactory architectural results could be obtained. But with open floor construction and the ties delivering the impact of the train directly to the concrete covering of the stringers, it seemed doubtful that the concrete would remain intact. This objection was overcome by changing to a through girder construction with solid floor. For the latter design the estimated cost was 1.6 times the cost of a plain steel structure.

Substituting entire reinforced-concrete arch construction for the type just discussed, satisfactory results in point of solidity, grace, beauty, monumental effect and the reduction of noise and vibration were obtained. The estimated cost was about 1.8 times that of a plain steel structure.

The substitution of a subway for the elevated structure would have been highly desirable except that the cost was estimated at four times that of an ordinary elevated structure.[1]

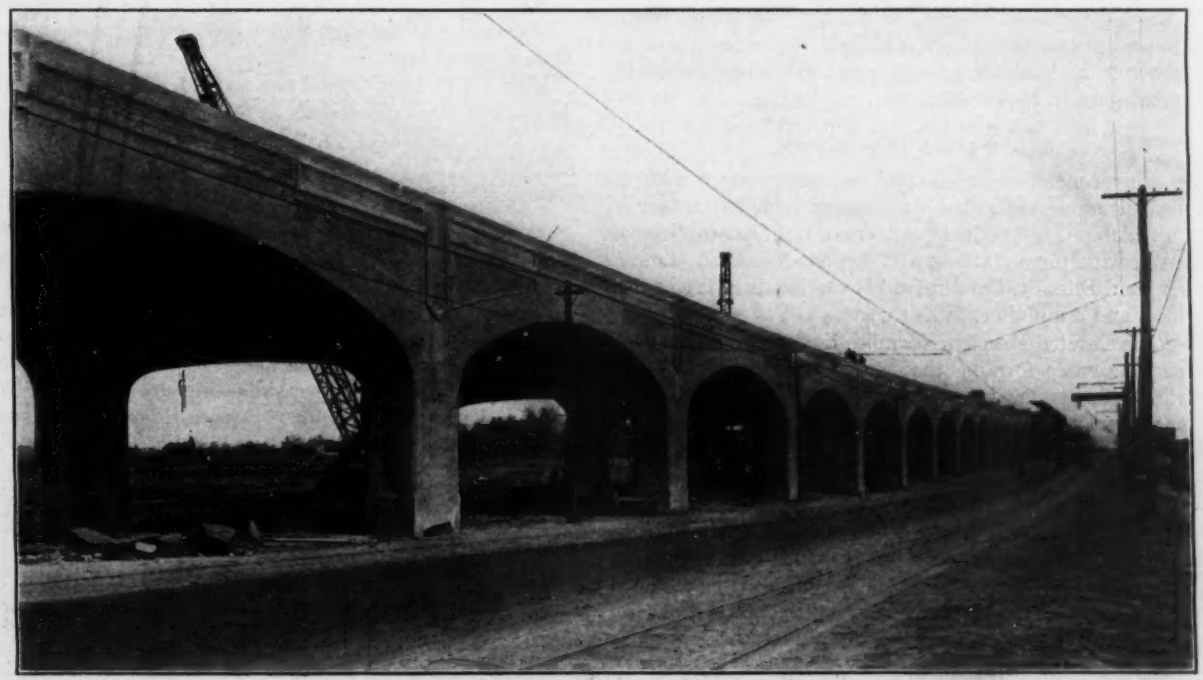


FIG. 1. COMPLETED STRETCH OF QUEENS BOULEVARD ELEVATED STRUCTURE

Colored tile is to be placed in recessed panels of the face. The station steel in the distance is to be cased in concrete

*Assistant Designing Engineer, Public Service Commission for the First District, 154 Nassau St., New York City.

[1]Topographical conditions made it necessary to extend a subway to the east beyond the point where the line leaves the Boulevard, increasing the relative cost for the length on the Boulevard from three times to four times that of the ordinary elevated.

In choosing between these different types the ornamental steel structure was abandoned because of its unsatisfactory detailing. For the remaining types the following tabulation shows the cost ratios:

RELATIVE COSTS OF DIFFERENT TYPES OF STRUCTURE

Ordinary steel elevated	1.0
Subway	4.0
Concrete-covered steel structure with solid floor	1.6
Concrete arch structure	1.8

Eliminating the subway because of its great excess of cost, it was decided that, the difference of cost between

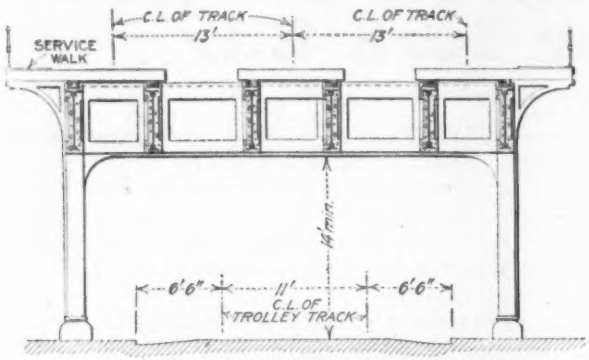


FIG. 2. SECTIONAL SKETCH OF VENEERED STEEL ELEVATED STRUCTURE

the other two types being so slight, the selection of the concrete arch structure was permissible because of the great gain in appearance.

General Description

This concrete-arch elevated railway viaduct with its domed arches supported on columns extends for over ¾ mi. along Queens Boulevard from Hill St. to Gosman Ave., with three stations, Rawson St.-Moore St., Lowery St., and Bliss St.-Carolin St. It carries three tracks, two local and a middle express track.

The elevated structure occupies the middle of the 74-ft parking space forming the central portion of the boulevard. Two trolley tracks occupy the center of the parking space and pass beneath the structure. Solid piers could not be used therefore either at the intermediate supports or at the abutments.

The over-all width of the structure is 44½ ft. A normal span length of 65 ft. was adopted, to fit both the 200-ft. block length and the 60-ft. width of cross-streets. Spans departing from the standard, in some cases down to a length of 52½ ft., were required at stations on account of the longer spans at the mezzanines.

A minimum clearance of 14 ft. over the trolley tracks is required. It was also desired that the height from street to station platforms should not exceed 30 ft., unless elevators or escalators were provided. This meant that the arches must be of very low rise; in fact with the flattest practicable arch the height to station platforms slightly exceeds 30 ft.

The general design of the structure is shown in Fig. 3.

Design of Arch

The ends of the arch are fixed, as the vaulted shape prevents any possibility of hinging. The investigation of stresses was made for a fixed arch by the strict method of elastic calculation, following closely the procedure given in Howe's "Symmetrical Masonry Arches." It was made on the assumption that each longitudinal strip acts independently of every other strip. Later, after the design was completed, the effect of the dome-shape was considered; the stresses were recalculated and proved to be not materially different from those found under the assumption that each longitudinal segment acts independently.

The design was made for a live-load consisting (for each track) of four 30,000-lb. axle loads, spaced 5, 10 and 5 ft.[2] Temperature change was taken as 30° F. rise or fall. Temperature stresses as well as the stresses due to shortening of the arch under load are included in the figures quoted later.

With a 4-ft. longitudinal rise on center line in the trial design and other dimensions as follows:

Span c. to c. of skewbacks	3 ft.
Rise transversely	2 ft. 4 in.
Crown thickness	2 ft.
Haunch thickness	4 ft.
Shape of arch	Circular segmental

The dead- and live-load stresses were found in general to fall within allowable limits, but the temperature stresses were excessive and the combined stresses prohibitive.

After some further rough trials it became evident that the rise of the arch and its flexibility must both be increased in order to reduce temperature stresses. The flexibility was increased by maintaining a constant section from crown to quarter point and increasing the thickness rapidly thence to the haunch. With a three-centered

[2]Dead-load includes weight of ballast at 100 lb. per cu.ft., concrete at 150 lb. per cu.ft., and the track. The stresses for a live-load of 2000 lb. per lin.ft. of track were in all cases less than those for the concentrated loads. The concentrated loads were assumed to distribute over 5 ft.—three ties—longitudinally and 12 ft. transversely. In spite of the use of ballast floor, impact was added, to the amount of 80% of the live-load, as computed from the Public Service Commission's formula: $I = 125 - \frac{1}{8}\sqrt{2000\,L - L^2}$; where L = loaded length of track. The stresses quoted further on include the impact effect.

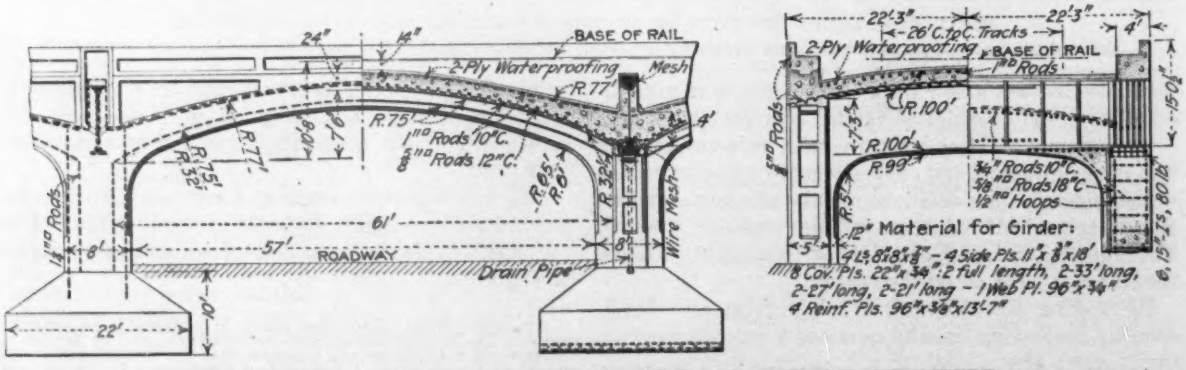


FIG. 3. TYPICAL ARCH SPAN OF QUEENS BOULEVARD ELEVATED STRUCTURE

segmental arch embodying these modifications, and with a longitudinal rise of 6½ ft. and a transverse rise of 18 in., the stresses seemed to come within practicable limits and the design was adopted. The reinforcing used comprises longitudinal rods 1 in. square spaced 10 in. c. to c., top and bottom; and transverse rods ⅝ in. square 18 in. c. to c., bottom only.

The stresses were first computed on the assumption of tension in the concrete, giving the following results:

	Crown		Quarter Point		Haunch	
	Top	Bottom	Top	Bottom	Top	Bottom
Concrete (max.)	+680	+500	+590	+150	+300	+430
Steel (max. comp.)	+10,000	+7,500	+9,000	2,200	+4,500	+6,500
Concrete (min.)	+40	—340	+230	—30	—260	0
Steel (min. compression or max. tension)	+600	—5,000	+3,400	—450	—3,900	0

As under these stresses the concrete would part in tension at the crown and the haunch under a combination of the most unfavorable conditions, the calculation was revised and the arch ring treated as a reinforced-concrete section subject to direct thrust and bending, with the usual assumption that tension is taken up entirely by the steel. On this basis the maximum stresses became:

	Crown		Quarter Point		Haunch	
	Top	Bottom	Top	Bottom	Top	Bottom
Concrete (max.)	+690	+500	+590	+150	+300	+560
Steel (max. comp.)	+10,000	+7,500	+9,000	+2,200	+4,500	+8,500
Concrete (min.)	+40	0	+230	0	0	0
Steel (min. compression or max. tension)	+600	—9,000	+3,400	—450	—7,000	0

All of these stresses are well within the elastic limits of steel and concrete, and, except for the maximum compression of the crown, within the Public Service Commission's working stresses (600 lb. concrete compression). As this case occurred only under the worst condition of load and temperature combination and with a large impact factor added to the live-load stress, it was considered entirely safe and no change was made on account of it.

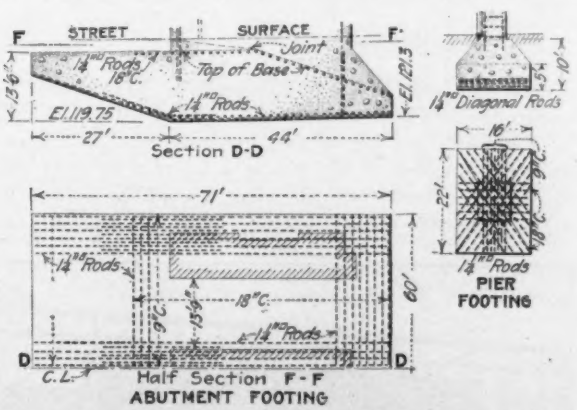


FIG. 4. NORMAL PIER FOOTING AND ABUTMENT FOOTING

Design of Columns

A cross-arch is sprung between the columns for appearance only, as an arch of sufficient strength to support the vertical component of the thrust at the haunch of the longitudinal arch proved entirely impracticable. A steel girder embedded in the concrete carries the load to the columns.

The dead-load thrust of adjacent arches balances at the top of the columns; the abutment of course is designed to resist the full thrust of the arch. The maximum live-load thrust of a single arch (adjacent arches unloaded) was assumed to distribute over *three bents*, so that each column was designed for one-sixth of the total live-load thrust from one arch with three tracks loaded. The highest column governed, and all columns, for appearance, were made of the same section.

A steel column, covered with concrete for architectural effect, was uneconomical, as the steel was concentrated near the center and therefore ineffective in resisting the bending stresses due to horizontal thrust, and the concrete added little strength. Furthermore, in order to prevent shrinkage, temperature and bending cracks, some reinforcement near the concrete faces would be required in any case. The steel columns would have been valuable during erection to support the girders so that they could be erected before any concrete was poured.

Type of Column Used

A concrete column with structural reinforcement in the form of a tower, with the verticals near the corners of the column, was economical. A detail, however, of a grillage to distribute the load of the completed structure over the concrete, and to support the weight of the girder during erection on the tower, proved unsatisfactory.

The only advantage of structural reinforcement over rods (which were used) was the support of the girder during construction. Practically the same results were obtained by pouring the concrete columns to the underside of the grillage, setting the steel, and then proceeding with the casting of the arch. In the end this proved to be an advantage to the contractor, for it was possible to pour the footings and columns while the girders were being detailed and fabricated.[3]

The columns are 5x8 ft. in section, with a rib 4 ft. wide projecting 1 ft. on the inside; they are reinforced with four 1¼-in. square rods at each end and ½-in. square hoops spaced 2 ft. apart vertically. On each side were placed four ⅝-in. square vertical rods to prevent temperature and shrinkage cracks. The reinforcement is proportioned for the bending with both ends of the column fixed.

On the basis of a maximum footing pressure of 2 tons per sq.ft. under direct load only, and a maximum edge pressure of 4 tons per sq.ft. (on sand and gravel), a footing 16 ft. wide and 22 ft. long of reinforced-concrete slab type with diagonal reinforcement was designed, as may be seen in Fig. 4. The original intention was to pour the entire footing as a monolith. This was found to be impracticable, however, so the footing was placed in two pourings, the top of the first lift being toothed in order to take care of horizontal shear.

Stations: Girder Construction and Mezzanine

At stations it was necessary to provide a mezzanine below the track for ticket booths and other facilities. A 65-ft. span was too short to give the required space, and the regular concrete-arch construction did not give sufficient clearance above the street to accommodate a mezzanine. Therefore the arch was replaced with steel girder construction 90 ft. in span.

Two serious problems arose in the station construction. The first was the support of the platforms along the sides of the regular concrete arch spans. In order to occupy as little as possible of the parkway, the structure was not widened, but the platforms were designed to

[3]About 75% of the columns were poured before any structural steel was delivered on the work.

overhang (Fig. 5). The outer edge of the platform was carried by longitudinal girders supported by brackets attached to the cross-girders, thus avoiding putting any additional load on the arches. Expansion in the arches produces no longitudinal movement, while the girders (due to stairways), it was necessary to provide heavy steel columns to support the main cross-girders, which receive all the load; the footings also were enlarged, to 21x26 ft.

The abutments at either end of the concrete structure, where the steel elevated railway adjoins, have to take

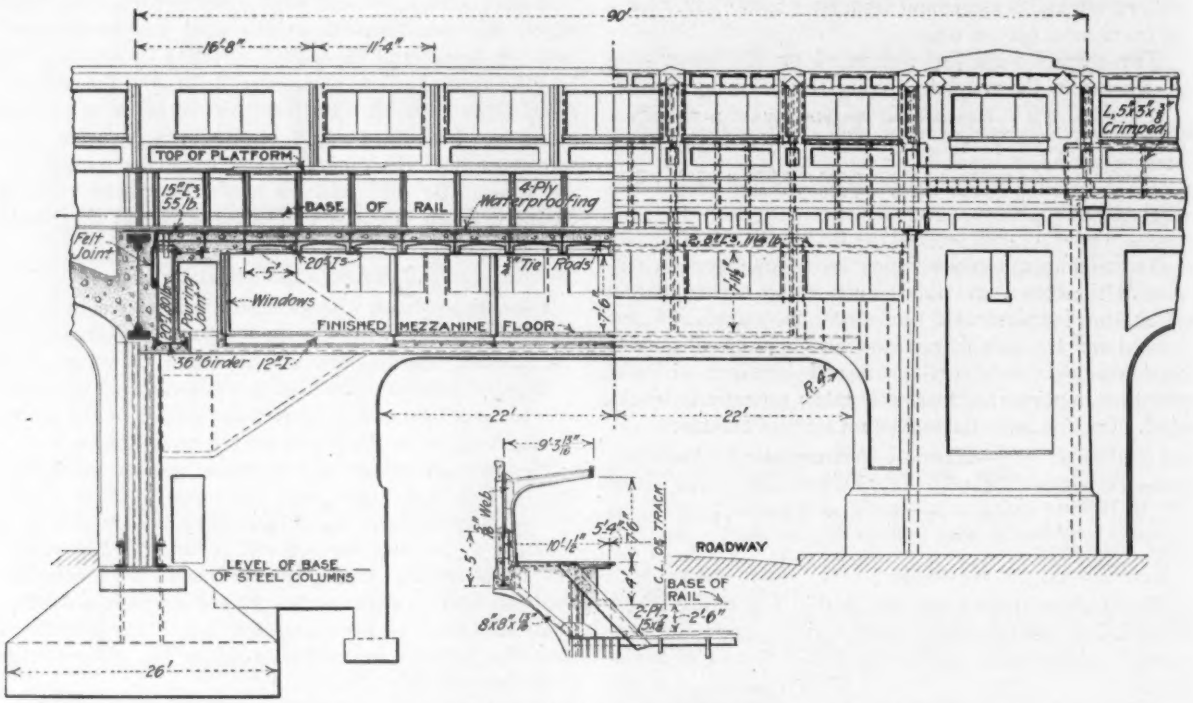


FIG. 5. STATION CONSTRUCTION ON CONCRETE ELEVATED RAILWAY

must expand and contract. The longitudinal girders and canopy columns are covered with concrete, thus maintaining a concrete exterior throughout. To prevent the concrete from cracking, expansion joints in the platforms were provided at the brackets. The expansions were so arranged that the joints in the concrete in general occur at the edge of a panel or pilaster.

The second problem was to provide for the arch thrust at the mezzanine spans. It was necessary to carry this thrust through the steel of the mezzanine and track floors. The average position of the thrust is slightly above the level of the former. The thrust is transmitted to the longitudinal struts by a vertical grillage of 20-in. I-beams attached at the top and bottom to horizontal girders in the track and mezzanine floors. It was first planned to attach the vertical grillage to the main cross-girders which support the longitudinal girders; the details, however, were very cumbersome and the actual stresses indeterminate. To avoid these difficulties the grillage is set clear of the cross-girders and its weight supported from the track floor. The thrust is carried into the beams by the concrete surrounding the grillage. The stress in the struts of the mezzanine floor is about 6000 lb. per sq.in. without temperature, and 14,000 lb. per sq.in. at maximum temperature; the shortening of the struts from the 8000 lb. per sq.in. temperature stress about equals the expansion of the steel due to the change in temperature, so that there will be no appreciable movement from temperature expansion.

Because of the increased load of the station span including the mezzanine, and the reduced section of the columns a maximum arch thrust of 4,088,000 lb. As passage for the trolley cars had to be provided, each abutment is built as a set of three heavy reinforced-concrete piers above ground connected at the top by a heavy reinforced slab to carry the thrust into the piers, and at the bottom by a single massive spread footing. Sufficient tension steel

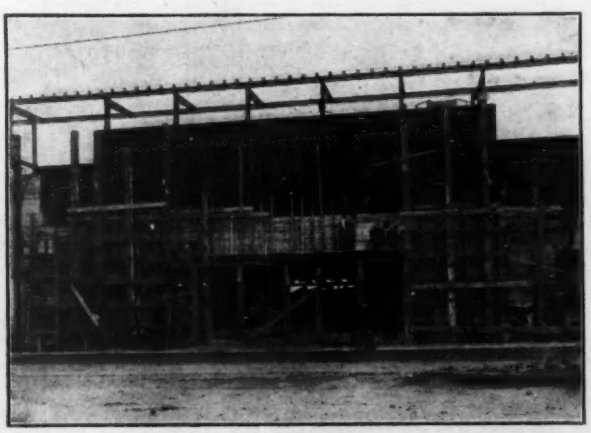


FIG. 6. STATION SPAN WITH FORMS FOR MEZZANINE PARTLY IN PLACE

is provided for the maximum bending stresses due to the arch thrust, and sufficient concrete area to take the shear. The toe of the footing is designed as a cantilever with a maximum toe pressure of 4 tons per sq.ft.

In arranging pouring joints the main objects considered were: (1) To secure stability, by making the joints

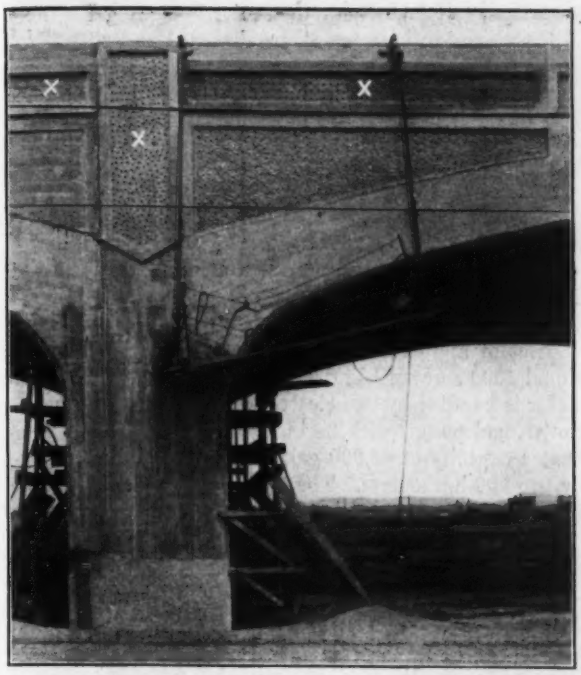


FIG. 7. FACE FINISH, QUEENS BOULEVARD ELEVATED RAILWAY

The arch at the right is tooled; the sunken panel is rough-point, the remainder of the face four-point work; the band around spaces "x" is rubbed. Colored tile is to be set in spaces "x." The arch at the left is untooled, showing the form marks.

normal to the line of strain, or by securing the sections against sliding through the use of dowel rods embedded about 3 ft. in each layer; (2) To arrange the joints where possible so that at the surface they came at edges of panels, etc., so as to be practically invisible.

The ordinary pier footings were divided into two pourings, as previously mentioned. The large footings of the abutments required more divisions. Here the work had to be done in longitudinal sections on account of the cantilever action. The base was, therefore, divided into six longitudinal sections, each containing about 300 cu.yd., equal to the normal daily capacity of the contractor's concreting plant.

The columns were poured in all cases to the underside of the grillage beams under the main cross-girders. In this pouring, skewbacks were molded for both longitudinal and cross arches, as shown in the section in Fig. 8.

The main arch was poured in three sections: first, the middle third, to bring the weight on the forms so that any settlement in them occurred before the arch was finally jointed; second, the haunches and the cross arch on both sides; third, the blocks covering the ends of the cross-girder. These blocks were poured before the arch centers were struck. The spandrel walls of the arch are separated from the arch vault by expansion joints made of a layer of elastic cement; these walls were poured last.

Finish of the Surface

In general the exposed concrete faces were dressed with a four-point tool used in a Boyer air hammer, with points set at the corners of a 1-in. square. Sunken panels on the underside of the ribs and in the spandrel walls are rough-pointed, using a single-point tool in an air hammer. Around the panels in the columns and arch ribs, and at other right-angle corners, a 6-in. draft margin is drawn with parallel cuts $\frac{5}{8}$-in. apart. The band around the colored tile on the face of the structure is rubbed with carborundum brick.

Forms and Plant

Wooden centers and forms were used exclusively on the work. The arch centers, designed by the contractor, are shown in detail in Fig. 8. It should be noted particularly that the lagging was supported on transverse rather than longitudinal trusses. The transverse curvature is uniform for all spans, so that the trusses could be used for any span by respacing them longitudinally.

The concrete was mixed at a central plant consisting of a 1-yd. Chain Belt rotary mixer driven by a 30-hp. a.c. motor. Cement was moved from the cement house to the platform of the mixer by a Robins belt conveyor driven by a 2-hp. a.c. motor, while the sand and gravel were handled with a 1-yd. Hayward clamshell bucket operated by a six-drum 52-hp. Maine Electric Co. hoist. The maximum capacity of the plant was 450 cu.yd. per day; the average capacity, about 300 cu.yd.

The concrete was handled for the most part in 1-yd. bottom-dump buckets and distributed on Koppel side-dump cars rebuilt for the purpose. These cars were moved in 5-car trains by three 24-in.-gage locomotives.

The concrete was handled and the forms moved and reset by two 15-ton Brownhoist locomotive cranes with 70-ft. booms. A derrick-car of 40 tons' capacity was used in setting the cross-girders. The heavy longitudinal girders at the station were handled by two 30-ton stiff-leg

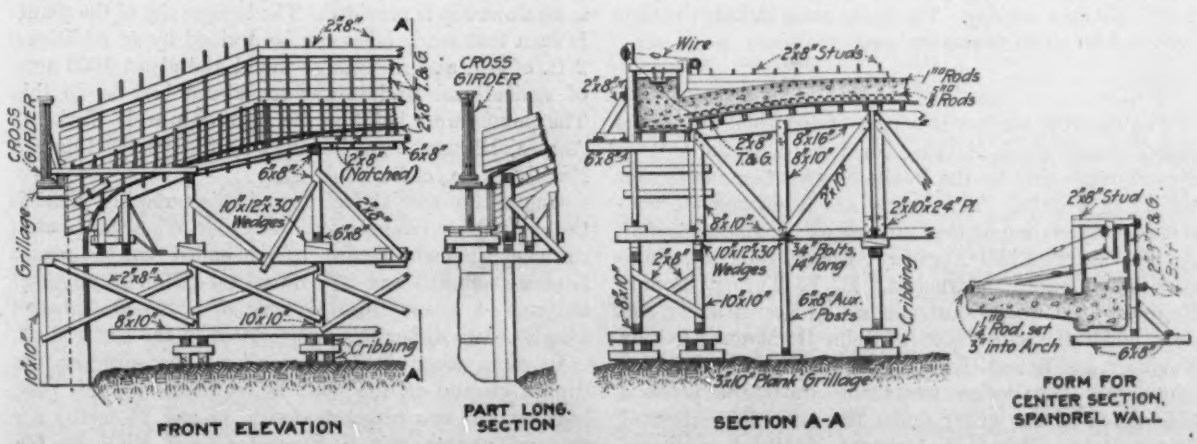


FIG. 8. FORMS FOR ARCH CONCRETING, QUEENS BOULEVARD ELEVATED

derricks. One stiff-leg derrick of the same capacity was used on the superstructure for erecting platform girders and light canopy material.

The plant for finishing the concrete consists of one 50-hp. locomotive boiler and one Ingersoll-Rand straight-line compressor with a capacity of 175 cu.ft. of free air per min. This supplies air for 23 hammers although usually only about 15 are working at one time. The staging for finishing the under side of arches is hung from the spandrel walls.

Progress

The first concrete in the footings was poured on July 1, 1913, and the first arch poured on June 19, 1914. The center of this arch was struck on Aug. 8. The contract provides that the concrete in the arches shall set 28 days before the centers are struck.

The concrete of the arches has all been placed and the concrete covering the steel of the stations is now being poured. About one-half of the surface has been finished.

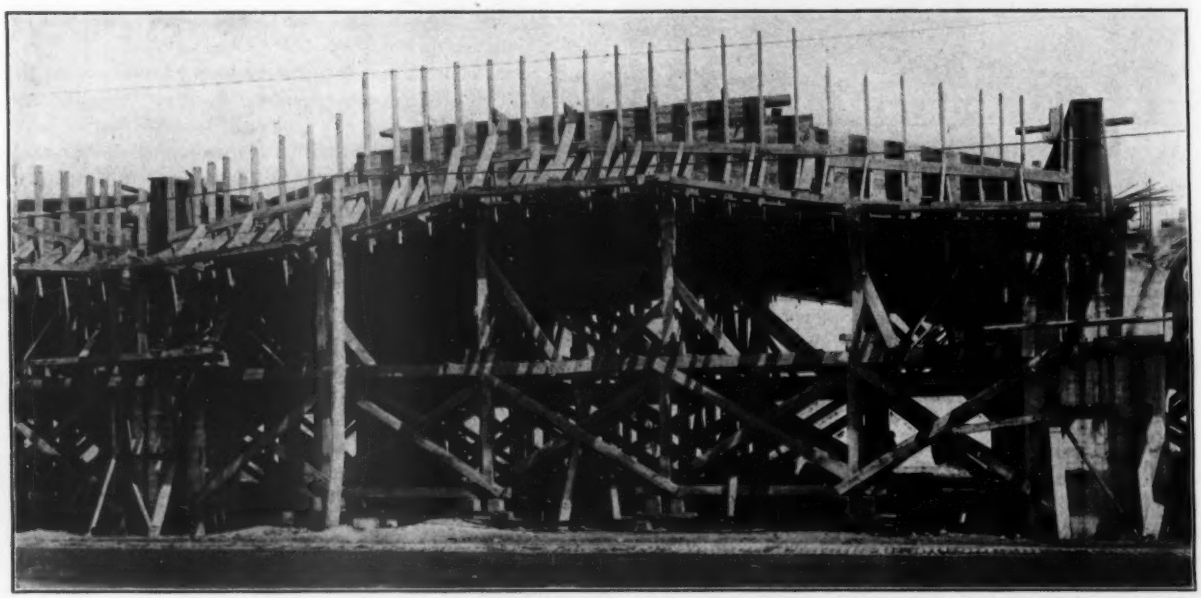


FIG. 9. ONE OF THE ARCHES OF THE QUEENS BOULEVARD WITH THE CENTERS IN PLACE

The daily rate for four-point work is from 75 to 90 sq.ft. per man per 8-hr. day, for rough-point work, about 60 sq.ft. The drafted margin is cut at the rate of about 30 lin.ft. per man per day. The above rates include the time required for shifting staging, etc.

Personnel

The concrete viaduct is a part of the Dual System of rapid transit which is being constructed by the City of New York, acting by the Public Service Commission for the First District. The design was made and the construction supervised by the Commission's engineering staff, Alfred Craven, Chief Engineer; Robert Ridgway, Engineer of Subway Construction; D. L. Turner, Deputy Engineer of Subway Construction. The construction was done under the supervision of John H. Myers, Division Engineer, and Robert H. Jacobs, Senior Assistant Division Engineer. The design was made under the personal supervision of the writer under the general direction of Sverre Dahm, Principal Assistant Engineer in charge of design, and A. I. Raisman, Senior Designing Engineer. The architectural treatment was designed under the direction of Squire J. Vickers, Designing Architect. The E. E. Smith Contracting Co. is the contractor for the elevated line to Corona, which includes the concrete-arch viaduct.

Variable-Speed Pumping Plant for a Drainage District

The pumping plant for the South Quincy Drainage and Levee District, in Adams County, Illinois, is designed to pump the drainage and seepage waters of a drainage district of approximately 6000 acres, to which an additional 4000 acres of hill drainage have to be added. This water is handled by two electrically driven double-suction centrifugal pumps with 36-in. discharge openings. They have a capacity of 60,000 gal. per min., and are operated by two 200-hp. motors. The layout of the plant is shown in Fig. 1. The current is supplied by an 11,000-volt line from the Mississippi River Power Co., which is transformed to 440 volts.

This plant has to pump all water falling in the district, as no sluiceway is provided. The topography of the county is such that many lakes can be drained by an additional 2 ft. of lift, and since these would add about 1000 acres of agricultural land it was considered best to do this. The pumps were built by the American Well Works, of Aurora, Ill., and were installed by the Farrar Pump & Machinery Co., of St. Louis, Mo.

One of the special features of this pumping plant is the chain-drive arrangement. In order to get the greatest efficiency the manufacturer recommended that the pumps be driven at 219 and 244 r.p.m. for different stages of water. To accomplish this change of speed, sprocket wheels of two different sizes are fitted to the motor shaft.

In some other plants of this kind the sprockets are slipped on and off the shaft as required. In this case, however, the two sprockets (with 30 and 27 teeth) are mounted permanently on the motor shaft, while the 59-

tooth sprocket on the pump shaft is arranged to slide longitudinally so that it can carry the chain for either one of the driving sprockets. This arrangement is shown in detail in the drawings in Fig. 2.

and allow the sprocket to be slid along the shaft. With a solid hub bored large enough to have the sprocket slide easily or freely, it was feared that it would be too loose for satisfactory operation. The split hub has a lock nut

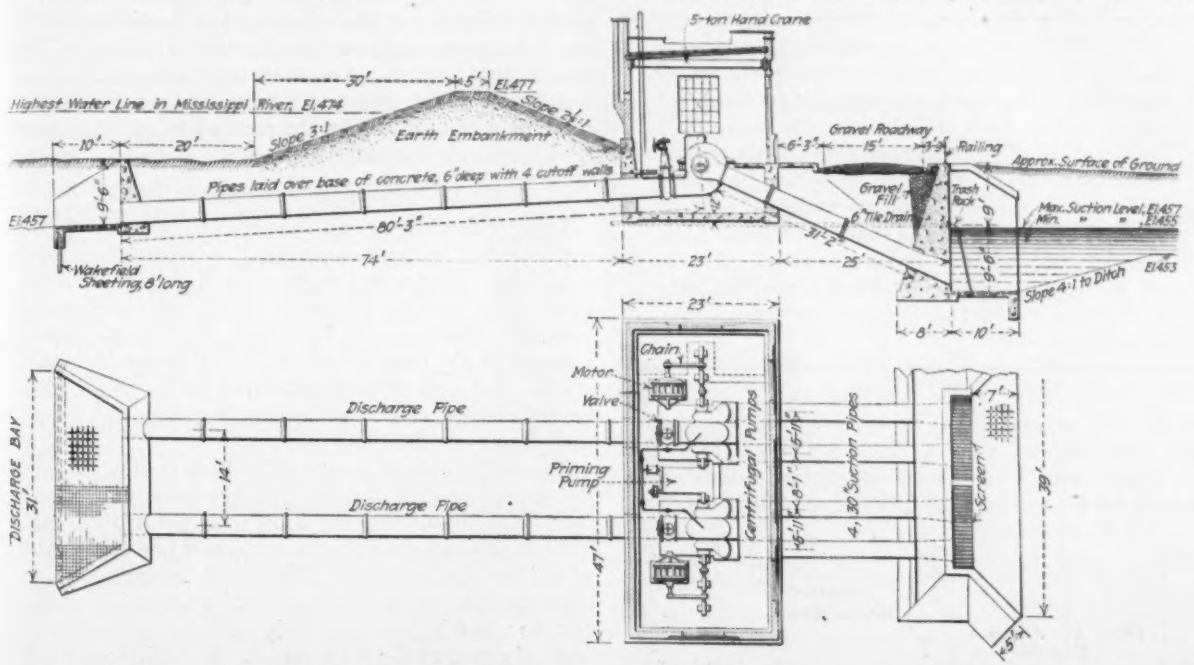


FIG. 1. PLAN AND CROSS-SECTION OF PUMPING PLANT FOR SOUTH QUINCY DRAINAGE AND LEVEE DISTRICT

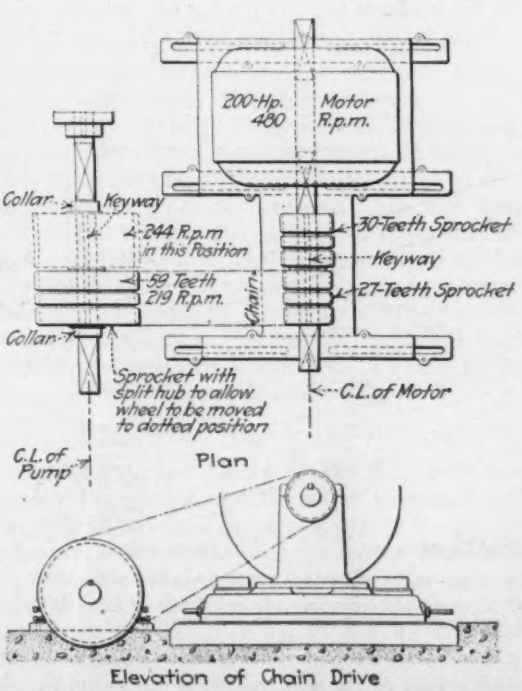


FIG. 2. VARIABLE-SPEED CHAIN DRIVE

The two pinions are fixed permanently to the driving shaft, and the chain is shifted from one to the other as required. The larger sprocket slides on the driven shaft

The larger (driven) sprocket has a split hub. At first it was intended to use a double-width sprocket, but the Morse Engineering Co., of St. Louis, advocated a single sprocket with a split hub. When shifting the chain from one driver sprocket to the other the bolt may be loosened with a key running the full length of the shaft, and is secured permanently in place.

To change the chain from one pinion to another a thin plate is employed, which is inserted under the chain after the center distance is shortened. Then it is easy to push the chain from one sprocket to another. To facilitate the work of shifting the chain, a pulley is fastened to the wall and has two clamps fitting over the chain. The two clamps are fastened close to the small pinion and its upper side. As soon as the chain is pulled out so that its lugs are clear of the sprocket, it is shifted to whichever side desired.

The drainage works and pumping plant for this district were designed by the Edmund T. Perkins Engineering Co., of Chicago. The construction was under A. H. Beitmann, who is in charge of the company's office at Quincy.

The Fees Allowed County Surveyors in Minnesota are based on a law dating back to 1867, when the Government section corners had been recently set and land surveying with a compass and chain, costing altogether less than $50, was more or less profitable. The fee then was $4 per day, and it is the same today, when a surveyor's outfit cost between $500 and $800. For a while this statutory fee was lost sight of and surveyors were able to earn a competence. A few years ago a certain county refused to pay a bill presented by a county surveyor. The case was taken into court and the old law dug up. According to N. Y. Taylor in a paper in the 1915 "Bulletin" of the Minnesota Surveyors' and Engineers' Society, there are few attractions to a county surveyor's office in that state until the old law is changed. A further reform Mr. Taylor proposes is that the notes and plats of all surveys made in a county by any surveyor be recorded by the county surveyor and the fees for the recording be paid by the county through the regular channels. The establishment and perpetuation of corners, as he says, is not a local and personal benefit, but a general and public interest. The conditions met with in Minnesota are by no means confined to that state.

Methods Used in Building the Rogers Pass Tunnel

SYNOPSIS—This 5-mi. double-track tunnel, now well advanced, will be by far the largest tunnel in north America, and is expected to be completed by the end of 1916. It is notable for the entirely new method of tunneling adopted. Intermediate headings are made practicable by driving from each end a pioneer tunnel parallel with, but distant from, the main tunnel and running crosscuts from the drift to the line of the main tunnel. Details are given of the tunneling work, timbering, concrete lining, ventilation and permanent track construction.

The 5-mi. tunnel of the Canadian Pacific Ry. at the Rogers Pass summit of the Rocky Mountains (having its east portal 80.5 mi. west of Field, B. C.) will be by far the longest tunnel on this continent, and will be large enough for a double track. It is being built not only to eliminate the present high-level summit and heavy grades, but also as a part of the extensive double-tracking plan to provide increased facilities for traffic. The tunnel project was described and illustrated in *Engineering News* of Apr. 2, 1914.

The tunnel will be 26,400 ft. long between portals, on tangent, and will have a grade of 0.95% throughout. In section it will be 29 ft. wide at rail level, with vertical sides and semicircular roof. The height will be 21½ ft. from base of rail to crown, or 23 ft. above subgrade. Construction was commenced in September, 1913, and the contract calls for completion within 3½ yr. (by Dec. 3, 1916). This necessitates rapid and steady progress, and for this purpose a novel method of driving the tunnel was devised by the contractors and approved by the railway company's engineers.

Two Pioneer Side Tunnels Employed

To drive this long tunnel from the two ends only would have involved a protracted period of construction, and in order to expedite the work it was highly desirable to attack it at various points by means of intermediate headings. Shafts as used for tunnels at moderate depths were out of the question for this great Rocky Mountain tunnel. Nor were there opportunities for the use of adits (or horizontal shafts), such as have been employed in exceptional cases of side-hill tunnels.

The method devised was to drive from each end a pioneer tunnel or drift parallel with the main tunnel, but about 50 ft. distant from it. From these two drifts lateral drifts or crosscuts are driven to the center line of the tunnel at intervals of 1,400 to 3,000 ft. From each of these points the main tunnel heading is driven, working usually in the upgrade direction. This method gives numerous points of attack on the main tunnel. The drifts serve for the removal of material by the mucking cars, the ventilation of the headings and the placing of pipes and cables clear of the tunnel construction work. The two pioneer drifts, however, are on opposite sides of the line of the tunnel, the drift at the east end being on the north side and that at the west end on the south side of the tunnel. Thus they cannot be extended to form a continuous passage, and each drift will be somewhat shorter than the half length of the tunnel.

These drifts and crosscuts were intended simply to facilitate the work on the tunnel, and their construction is entirely at the contractors' expense. It was not intended to use them permanently, and in fact if loose ground

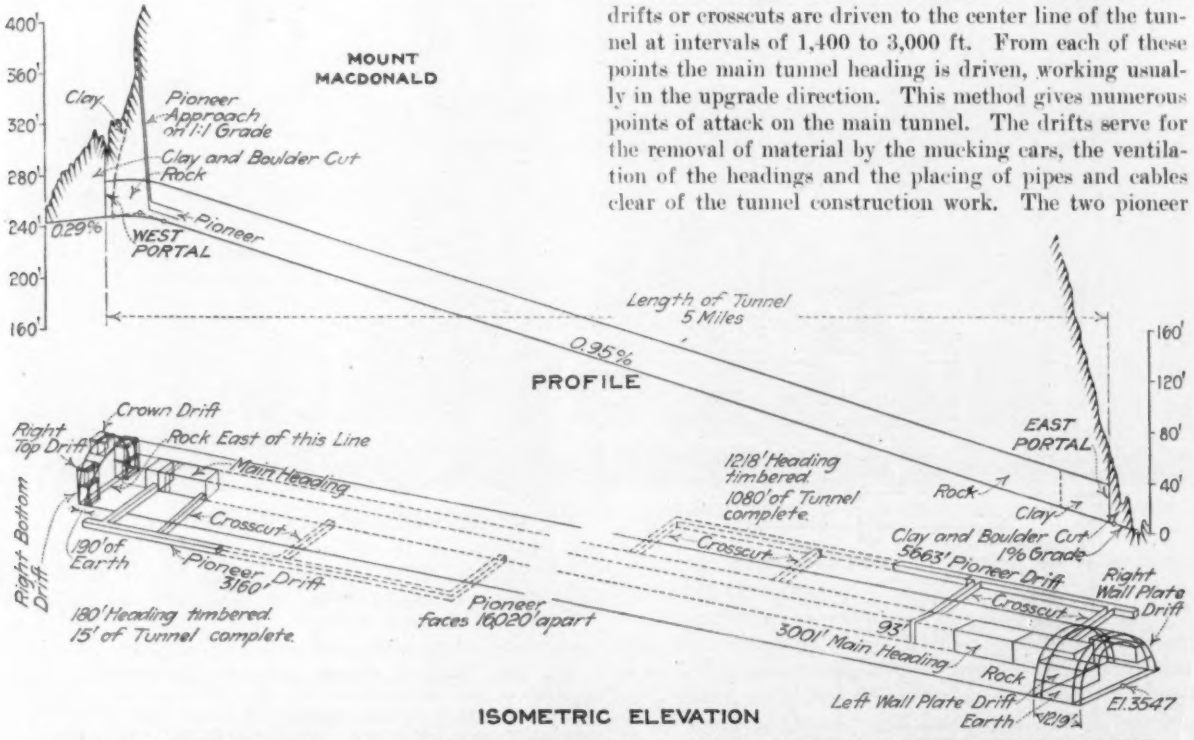


FIG. 1. ISOMETRIC SECTION AND PROFILE OF THE ROGERS PASS TUNNEL, CANADIAN PACIFIC RAILWAY

The tunnel is being driven from intermediate headings reached by crosscuts from two pioneer drifts or tunnels parallel with but 50 ft. distant from the main tunnel, and on opposite sides of the tunnel line

is encountered they may be filled with concrete after the completion of the work. There seems to be a possibility, however, that they may be utilized in the ventilating system for the finished tunnel, as noted further on.

The nearest approach to a previous use of this system in driving long tunnels was in the construction of the first Simplon Tunnel in Switzerland. A small drift was run on one side parallel with and a little in advance of the main tunnel, and connected with it by lateral drifts. But in this case the main tunnel was for single-track only, and the parallel drift was intended for use temporarily as a part of the ventilating system and eventually to be enlarged to form a second single-track tunnel. Its enlargement for this purpose to meet the traffic requirements of the Simplon route was begun in 1913 and is now under way. The rate of progress and cost of work on the great Canadian tunnel are considered to have amply justified the driving of the auxiliary construction drifts.

The method of construction is shown clearly in the isometric drawing, Fig. 1. The pioneer drifts and crosscuts are of rectangular section, 8 ft. wide and 6 ft. 6 in. high. The main or central heading, also rectangular, is 11 ft. wide and 9 ft. high. Working from each portal, this main drift is being enlarged to the full tunnel section shown in Fig. 2.

The object aimed at by the novel method adopted for the construction has been attained. This is shown by the fact that the steam shovels (one working at each end) have advanced at the rate of 1,636 ft. per month for the last three months. On July 25 the two pioneer tunnels were stopped when they were 5,202 ft. apart. Crosscuts were made to the center or tunnel headings at these points, and the remaining mile of center heading will be driven and the drilling for enlargement completed before the mucking shovels reach these last crosscuts.

Up to Aug. 19 there had been driven 10,740 ft. of pioneer heading at the east end and 8,870 ft. at the west end. There were 7,366 ft. of main heading driven at the east end and 8,352 ft. at the west end. On that date the most advanced headings were only 4,677 ft., or 0.89 mi. apart. The shovels on the enlargement of the tunnel to full section have advanced 5,612 ft. at the east end and 4,300 ft. at the west end, leaving 16,488 ft. to be covered. Of the concrete lining at the ends about 1,600 ft. has been completed. The progress up to Aug. 1 was given in *Engineering News* of Aug. 12, 1915.

Driving the Drifts and Headings

To save time and excavation the pioneer drifts were started considerably above the grade of the tunnel heading. The drift at the east end starts about 50 ft. above grade and is practically level, so that the first two crosscuts slope down to reach the main heading. From the point where the drift strikes the same elevation as the tunnel it rises with the tunnel grade of 0.95% and the crosscuts are level. The drift at the west end starts about 130 ft. above grade and descends on a sharp incline of 50% grade for 230 ft., which brings it to the elevation of the tunnel heading. Beyond this the drift is on the same grade as the heading and the crosscuts are level. At first the muck cars were hauled up this 50% incline by cables. After the full tunnel section had reached the first crosscut the cars were diverted through this into the tunnel (starting Apr. 29), as described below, thus eliminating the use of this incline.

Drainage of the face of the west pioneer drift (working down grade) is handled by a pump at the face and two intermediate station pumps. It is not expected that much water will be encountered in the rock, but in the loose material at the ends it has been necessary to pack hay behind the timbering of the side-wall drifts to prevent washing in of soil.

The muck from the pioneer drift and the various points on the main heading is hauled in ½-yd. cars through

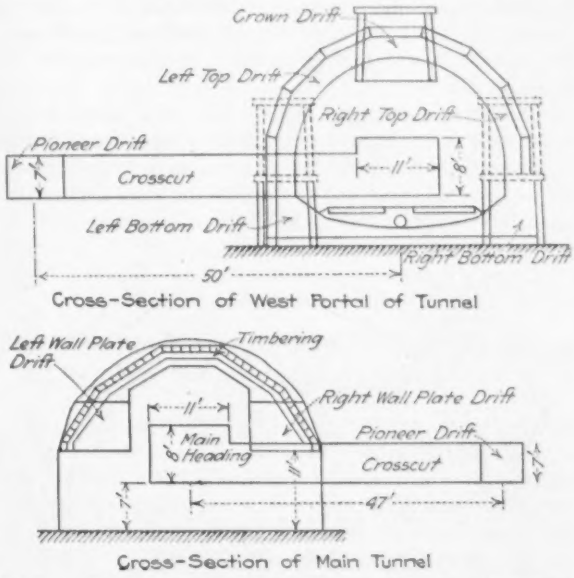


FIG. 2. METHODS OF DRIVING THE ROGERS PASS TUNNEL FROM THE PORTALS

the drift (and crosscuts) to the nearest crosscut which has been reached by the full tunnel section. Here the cars are diverted to enter the tunnel and dump their contents into the 12-yd. standard-gage cars which serve the mucking shovels at the face. The shovels are air-operated machines of about 40 tons with 1- and 1½-yd. buckets.

The ½-yd. cars were at first hauled by mules, but narrow-gage compressed-air locomotives are now used. The larger cars are hauled by 10- and 12-ton compressed-air locomotives working between the face and the sidings for loaded and empty trains. These trains are handled by 40-ton six-wheel steam locomotives between these sidings and the outside dumping places. At the east and west ends the muck is dumped to form the long approach embankments to the tunnel.

Drilling and Power Plant

In driving the pioneer drifts at the east end four Leyner drills are mounted on a bar. These put in about 24 holes 6 ft. deep, 1¼-in. top and 1⅛-in. bottom diameter. Each hole is loaded with from 4 to 8 lb. of 50% dynamite. Three to six rounds are usually fired per day of 24 hr. (three 8-hr. shifts). Similar drills are used in the center bench, drilling rounds of 26 holes 6 ft. apart, or 225 lin.ft. per round. These sections or "rings" are shot off for muck as required.

The compressed-air plant at each end includes two compressors supplying air at 125-lb. pressure for the drills and shovels and one supplying it at 950-lb. pressure for the locomotives. The latter has a capacity of 750 cu.ft. of free air per minute, while the two others have capacities

of 1,000 and 2,000 cu.ft. Each locomotive has storage capacity sufficient for a 1-mi. run with a load, and the 1½-in. high-pressure main is extended into the tunnel, providing air stations about 2,000 ft. apart.

Driving the Portal Headings

For driving the full tunnel section from the portal in the east end two side drifts, 7x8 ft., were driven, with floors about 18 in. below the level of the wall plates for the roof centering. These were connected by a curved crown drift so that the centering could be placed, and the inclosed "core" then removed. The construction is shown in Fig. 2.

For similar work at the west end the two timbered side drifts were at grade and were about 6 ft. wide and 8½ ft. high. Upper-side drifts, 6x7 ft., were driven above these, the muck being delivered through the roof timbering of the lower drift into cars standing in that drift. A crown drift 8 ft. wide and 9 ft. high was also driven on the center line (Fig. 2). The material between the crown and upper-side drifts was then broken through and the timbering placed outside the line of the future concrete lining.

Timbering and Concrete Lining

With the exception of several hundred feet of clay and glacial drift at each end, the drifts are in solid rock, which is expected to continue throughout the entire length. It is mainly slate, schist and quartzite. No timbering is required in the rock excavation.

For the end portions, which are in loose material and are timbered, a concrete lining is required. This is 30 in. thick and 483 ft. long at the west end and 27 in. thick and 1,288 ft. long at the east end. In addition, on account of the spalling of the rock, some concrete lining may be required in the solid-rock section at the west end. Typical sections of the tunnel, with its timbering and concrete lining, are shown in Fig. 3. The timbering in the glacial drift consists of a semicircular roof arch supported on 12x16-in. posts. The timber sets are usually spaced 18 in. c. to c., but are set close where the material is loose and contains water. The arch has five or seven segments, usually single, but sometimes double.

An interesting feature is that for the lined section in loose material the heavy footings of the side walls are braced by reinforced-concrete struts 20 ft. apart. These are 18 in. wide, 24 in. deep at the ends and 18 in. at the middle. They are formed monolithic with the footings and with the longitudinal concrete slab for the support of the track drain.

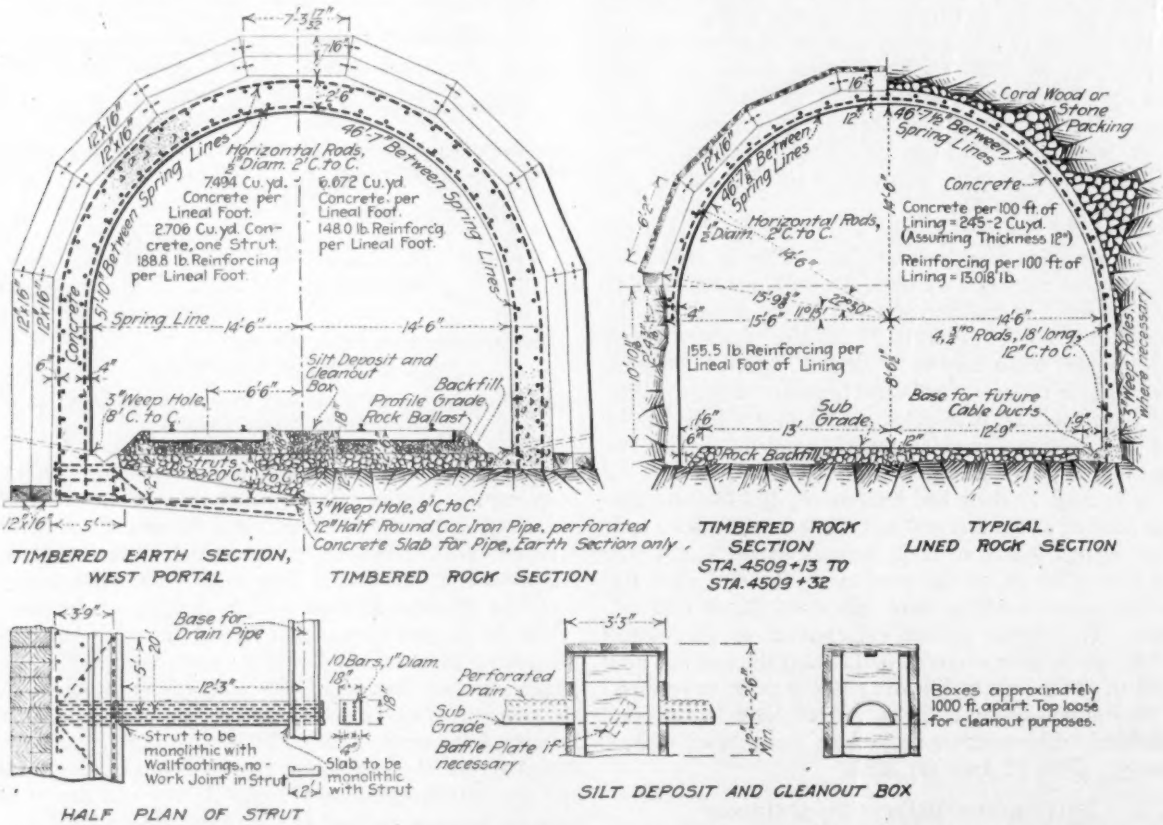

Horizontal Rods, ⅝" Diam. 2' C. to C.
7494 Cu. yd. Concrete per Lineal Foot. 2706 Cu. yd. Concrete, one Strut. 188.8 lb. Reinforcing per Lineal Foot
6.672 Cu. yd. Concrete per Lineal Foot. 148.0 lb. Reinforcg. per Lineal Foot.
46'-7" Between Spring Lines
Spring Line
14'-6"
Silt Deposit and Cleanout Box
Backfill
Profile Grade
Rock Ballast
3" Weep Hole, 8' C. to C.
12" Half Round Cor. Iron Pipe, perforated
Concrete Slab for Pipe, Earth Section only
TIMBERED EARTH SECTION, WEST PORTAL
TIMBERED ROCK SECTION
Cord Wood or Stone Packing
Concrete
Concrete per 100 ft. of Lining = 245·2 Cu. yd. (Assuming Thickness 12")
Reinforcing per 100 ft. of Lining = 13,018 lb.
155.5 lb. Reinforcing per Lineal Foot of Lining
4, ¾" ø Rods, 15' long, 12" C. to C.
Sub Grade
Base for Future Cable Ducts
Rock Backfill
TIMBERED ROCK SECTION STA. 4509+13 TO STA. 4509+32
TYPICAL LINED ROCK SECTION
Base for Drain Pipe
10 Bars, 1" Diam.
Strut to be monolithic with Wallfootings, no Work Joint in Strut
Slab to be monolithic with Strut
HALF PLAN OF STRUT
Perforated Drain
Sub Grade
Baffle Plate if necessary
SILT DEPOSIT AND CLEANOUT BOX
Boxes approximately 1000 ft. apart. Top loose for cleanout purposes.

FIG. 3. CROSS-SECTIONS OF THE ROGERS PASS TUNNEL, SHOWING TIMBERING AND CONCRETE LINING

The concrete for the tunnel lining is mixed by concrete mixers on the outside and loaded into cars, which are hauled into the mouth of the tunnel by a dinkey. A cable from a hoisting engine hauls the cars up to the work. The concrete is dumped into the forms for the side walls from an elevated track on scaffolding, but is shoveled by hand into the arch form above the level of the scaffolding. Timber forms are being used, as the amount of concrete does not justify steel forms.

For ventilating the working places in the center bench and main tunnel there is a large blower installed in each pioneer drift. This is set up to draw pure air through the drift and discharge it into the center bench through a crosscut ahead of the center-bench drills.

For ventilating the faces of the pioneer drift and advance heading two exhaust fans of the Connersville type (with 12-in. wood-stave pipe) are used, placed usually near a crosscut. From this point the exhaust pipe is carried to within 25 ft. of each face. A gate is provided so that the fan can exhaust from either face as required. Only one of these fans is worked at a time, so that one can be moved while the other is in service. The fan is started at each blast, and the men can return to work in from 15 to 25 min. By opening the 1½-in. air line for the drills the smoke and fumes can be cleared away from the face so that they will be removed by the exhaust.

At the portal of each pioneer drift there has been installed a multivane fan 5 ft. in diameter, which forces air into the drift. This air is diverted into the main tunnel whenever required. For instance, after shooting in the enlargement to full tunnel section the air is turned in at a cross-drift beyond the point of shooting. In this way the smoke and gases are cleared out of the tunnel very rapidly.

The compressed-air pipes, ventilating pipes and telephone lines are laid through the pioneer drifts and crosscuts, so that they are not affected by blasting and do not interfere with the work of the mucking shovels in the full tunnel section. No electric lighting is used. The men carry acetylene lamps and acetylene headlights are applied on the mucking shovels and the locomotives.

Working Force and Staff

The force per 8-hr. shift in the pioneer drift is as follows: Four drillers (4 drills at each end), 2 helpers, 8 to 10 muckers, 1 nipper, 1 driver, 1 shift boss, 1 to 4 mules and about 40 cars.

The force per 8-hr. shift in the advance heading of the main tunnel is as follows: Three drillers (3 drills at each end), 2 helpers, 8 to 10 muckers, 1 nipper, 1 driver, 1 shift boss, 1 mule, and 60 to 80 cars (according to the length of haul). In addition, there is the following force on each 10-hr. shift to cover the two small drifts in each end: One assistant superintendent, 1 walking boss, 2 blacksmiths, 2 blacksmith helpers, 12 laborers, 10 dump men, 6 teamsters, 14 mules, 1 hoistman and 7 pipe-fitters, carpenters, mechanics, etc.

The center bench of the main tunnel is worked in 8-hr. shifts with 7 to 10 drills, 7 to 10 helpers, 5 to 10 muckers, 1 shifter, 1 driver and 1 mule.

The completion of the main tunnel to full section is worked in 10-hr. shifts, with the following force per shift: One driller, 1 helper, 1 to 2 powdermen, 1 engineman and 1 craneman on the mucking shovel, 10 to 12 pit men at the shovel, 1 general foreman, 1 foreman and 12 to 15 laborers for the dump and track work, 2 compressed-air dinkey locomotives with 2 enginemen and 2 brakemen, 1 steam locomotive and 1 engineman, 1 fireman, 1 conductor and 2 car-repair men.

The contractors for the tunnel are Foley Bros., Welch & Stuart, of Winnipeg, and A. C. Dennis is their superintendent. The work is under the direction of John G. Sullivan, Chief Engineer, Canadian Pacific Ry., and W. A. James, Engineer of Construction.

In the finished tunnel the tracks will be spaced 13 ft. c. to c., and will be laid with 85-lb. rails on wood ties embedded in 18 in. of stone ballast. Beneath the ballast will be 12 in. of rock fill, the floor of the excavation being carried down 12 in. below subgrade. A semicircular drain of corrugated iron will be laid in the rock fill beneath the tracks, the pipe being perforated for the admission of water. In general it will be laid on the rock floor, but in the loose material at the ends the pipe will be laid on a concrete slab of channel section, supported by the concrete struts between the footings of the side walls (Fig. 3).

The plans for the tunnel project originally contemplated the introduction of electric traction in the tunnel and over a distance of about 25 mi., and no artificial ventilation was then considered necessary. This feature of the project has been abandoned and a ventilating system is to be installed to avoid possible troubles due to smoke and gas in the 5-mi. bore. The definite plans for this system have not been determined, but possibly it may be found practicable and economical to utilize the construction drifts, with their lateral connections to the tunnel, as part of the permanent ventilating system. These drifts, as described in the foregoing, are outside of and parallel with the tunnel, but do not form a continuous passage.

Interior Temperature Changes in Mass Concrete

In the current *Proceedings* of the American Society of Civil Engineers, C. H. Paul and A. B. Mayhew, of the United States Reclamation Service, in a paper entitled "Temperature Changes in Mass Concrete," have described in detail some experiments made at the Arrowrock Dam in Idaho to determine the temperature changes in the interior of mass concrete as affected by outside temperatures and conditions of setting. The final conclusions of the tests are as follows:

1. Large bodies of concrete deposited rapidly during the summer season develop a temperature of from 90 to 95° within a period of about 30 days, and maintain nearly that temperature for several months.

2. In the case of concrete 1 ft. from an exposed face there is a daily variation in temperature of about 2° when the daily variation in the temperature of the air is about 50°.

3. In the case of concrete 2 ft. from an exposed face there is a daily variation of less than 1° when the daily variation in the temperature of the air is about 50°.

4. In the case of concrete 3.5 ft. from an exposed face, no daily variation in temperature is apparent when the daily variation in the temperature of the air is about 50°.

5. The seasonal variation in the temperature of concrete 3.5 ft. from an exposed face is about 32° when the seasonal variation in the mean daily temperature of the air is about 75°.

6. The seasonal variation in the temperature of concrete 10 ft. from an exposed face is about 12° when the seasonal variation of the mean daily temperature of the air is about 72°.

7. The seasonal variation in the temperature of concrete 20 ft. from an exposed face is very little, and after the effect of "setting heat" has once been overcome, the change in temperature of concrete at that distance from an exposed face is so slight as to be negligible under ordinary conditions.

8. The effect of the "setting heat" in concrete 20 ft. or more from an exposed face is felt for several years after the concrete is placed. Concrete near the center of mass of a large dam probably retains some of this "setting heat" for 5 yr. or more after placing.

9. Contraction joints are desirable in all large concrete structures, whether or not they are to be exposed to wide variations of outside temperature, except possibly when all construction may be carried on during cold weather and when the variation of outside temperatures, after completion, will be slight.

10. Changes in volume due to setting, hardening and seasoning of concrete are more important, ordinarily, except very close to exposed faces, than changes in volume due to the influence of daily or seasonal variations in outside temperatures.

Laying Creosoted Wood-Block on 5 Per Cent. Grades

By J. R. West*

In connection with the Bell St. warehouse of the Port of Seattle there are certain elevated roadways on grades varying from level to 5¼%. Generally these roadways are carried on timber structures, and in order to reduce the dead-load as much as possible it was decided to try creosoted-wood block, although the grades are rather steep for this type of pavement. On the level portions the

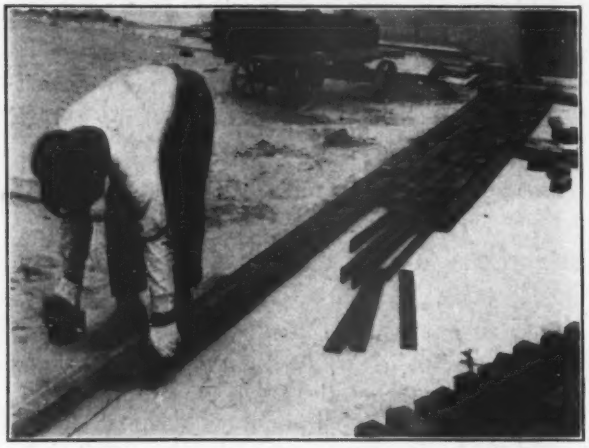


FIG. 1. LAYING CREOSOTED WOOD-BLOCK PAVEMENT ON A 5% GRADE

Showing expansion transverse joint and wood-strip joint fillers

blocks were laid with tight joints, but on the grades creosoted-wood spacing strips ½ in. thick and 2½ in. wide were laid in all joints. The blocks were 3 in. depth, which gave a notch ½ in. wide and ½ in. deep between the rows of blocks to afford foothold for horses.

The blocks were cut from sound Douglas-fir lumber containing an average of at least five annual rings per inch, and at no point less than four annual rings per inch; and it was required that the lumber should consist of at least 40% of dense summer wood. Sound knots not over 1½ in. in diameter were permitted, but no knots were allowed to show on the upper surface.

The blocks were required to be air seasoned before being treated. The treating was done by boiling, from 16 to 18 lb. of creosote per cubic foot of wood being forced into the blocks. This amount was then reduced to about 10 lb. by expansion and vacuum. The preservative was a pure coal-tar creosote oil. The pressure was not allowed to run higher than 100 lb. per sq.in., and the oil temperature was about 180° F.

In order to protect the timber structure underneath the pavement from moisture, two layers of Barrett's No. 2 roofing felt mopped with tar were laid as a waterproofing membrane. This membrane was carefully turned up at the edges of the pavement and around the drain inlets, and care was taken to insure thorough waterproofing of the foundation.

The blocks were laid directly on the felt membrane, without sand cushion. This, however, appeared to be a mistake, for the blocks absorbed enough water after the first hard rain to cause a considerable movement at some of the expansion joints, and the blocks being embedded in the tar apparently ruptured the waterproofing membrane at various points, where it was found to leak.

After the blocks were laid and driven tight a joint filler consisting of a dry mixture of one part portland cement and three parts fine sand was used. It was heated to a temperature of about 200° F., spread over the pavement and swept into the joints, the excess being allowed to remain on the surface.. Expansion joints 1½ in. wide were placed along the curbs and across the roadways every 80 ft. These were filled to one-half the depth with a high-grade asphalt mixed, while the asphalt was hot, with 20% by weight of portland cement and sand in equal parts. This type of expansion joint has, however, not proved satisfactory, because the sand and cement settled to the bottom and formed a mass so hard that the joints failed entirely to allow compression. It has been necessary to chop out these joints and replace the filler with asphalt. After the joints were partly filled as noted, dry sand was spread over the pavement to a depth of about ½ in. and allowed to remain there. A considerable portion of this sand was worked into the joints by traffic. The remainder was removed after the rainy season had set in, for it acted like a sponge, holding the water and not permitting the blocks to dry out. The pavement was not slippery either before or after the sand was removed; in fact, teamsters who have used this pavement in hauling heavy loads have stated that it was easier to start a load on a 5% grade on this pavement than on a brick pavement.

Outside of the minor difficulties which have developed, and which are not inherent with this type of pavement, the experiment has been very satisfactory and certainly

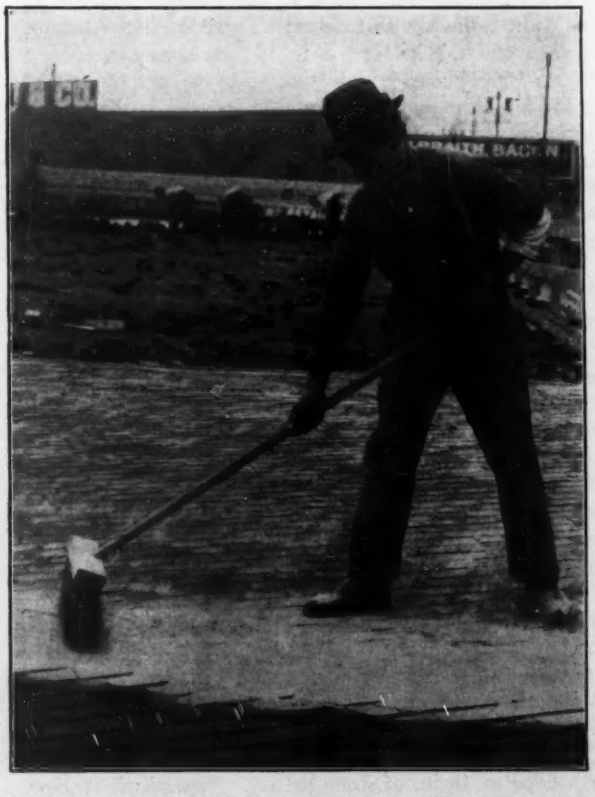


FIG. 2. FILLING UPPER HALF OF JOINTS WITH HOT SAND AND CEMENT

*Chief Engineer, Port of Seattle, Washington.

demonstrates that a creosoted-wood-block pavement can be used on steeper grades than have been thought safe.

This pavement, including the waterproofing, was laid at a cost of $1.44 per sq.yd. The contractor was the Ferguson Construction Co., of Seattle, and the blocks were furnished by the Pacific Creosoting Co., which has assumed the maintenance of the pavement for a period of five years and has offered many valuable suggestions.

The plans and specifications were drawn and the construction inspected by the engineering organization of the Port of Seattle.

Plans for the Hetch-Hetchy Construction Railroad

The site of the Hetch-Hetchy dam, to be built on the headwaters of the Tuolumne River in California to store water for the future supply of San Francisco, is in a deep valley of the Sierras, at an elevation of nearly 4,000 ft. above sea level and distant nearly 70 mi. from the nearest railway line.

A wagon road exists to the side of the dam, but it has been determined that the amount of material to be handled during construction of the dam will be so large that it will pay to build a railway to the dam site, even though the entire cost of the railway line has to be included in the cost of the dam.

Under direction of City Engineer M. M. O'Shaughnessy surveys have been made and plans and specifications have been prepared for the construction of a railway line 67 mi. in length, and bids for the work are now being advertised for. The line follows the general course of the Tuolumne River most of the way, but as the river runs much of the distance through a narrow cañon with walls nearly vertical, it was not practicable to follow the river closely, and the location is on an elevated bench at some distance from the river.

Location of Road

The railway branches from the Sierra R.R. at an elevation of 935 ft., drops to 589 ft. to cross a valley and then ascends with a somewhat broken profile to a maximum elevation of 5,064 ft., finally dropping down for 6 mi. on a continuous 4% grade to the dam site. Most of the climb going to the dam is made on a 3.4% grade. Adverse grades for all eastbound traffic and for part of the westbound traffic are compensated for curvature at the rate of 0.04 of 1% per degree of curvature.

As the road runs through an exceedingly rough and broken country, is to be operated for freight traffic chiefly and is a construction railroad rather than a permanent line, sharp curvature has been freely used in location. The sharpest curve is 30°, and curves of 18 to 26° are very numerous. It is intended to lay guard rails on curves sharper than 12°.

Notwithstanding the fact that the railway is a construction line, complete detailed specifications have been prepared for the construction. The work involves an estimated quantity of over 1,000,000 cu.yd. of excavation, more than half of it in rock, about 6,700 lin.ft. of corrugated-metal culverts, 40 mi. of fencing, laying 69 mi. of track, including yards and sidings, and nearly 500,000 ft. of lumber in trestles.

An unusual classification is adopted for excavation, the material being classified as granite, solid rock, soft rock and earth, although alternative bids are invited for the entire excavation without classification. The reason why granite and solid rock are classified separately is that the Sierra granite, of which over 160,000 yd. is included in the estimate, is a very much harder rock to drill and break up than the slates, sandstones, shales and other solid rocks occurring along the line. The granite classification includes all masses of granite excavated of 1 cu.yd. or more. The solid rock includes all pieces of rock of 1-cu.ft. or more volume. Soft rock includes all cemented materials that can be removed without blasting.

The roadbed is to be 16 ft. wide at subgrade, and slopes on embankments are specified as 1½ horizontal to 1 vertical. The limit of free haul is 400 ft., and overhaul is paid for at the rate of 1c. per cu.yd. per 100 ft. excess.

Metal Culverts

For galvanized-metal culverts, alternative bids are asked on "pure iron" and on "copper-bearing steel," the first being specified to be 99.84% pure iron and the steel to have between 0.20% and 0.30% of copper. The galvanizing is to be done with not less than 2 oz. of prime spelter per square foot. (1 oz. on each side), to be determined by the lead acetate method. Corrugations are to be spaced between 2½ and 3 in. and ½ to ⅝ in. deep. The longitudinal joints must have a lap of at least 1½ in., with rivets in each outside groove, and the transverse joints are to have rivets not more than 6 in. apart.

Concrete pipe of a 1:2:4 mixture may be used as an alternative to metal pipe.

Details of Trackwork

Ballast of broken stone, of gravel or of selected materials from the excavation may be used, and separate bids are invited for the different classes. The ballast is to be 6 in. deep under the ties, must extend 6 in. beyond their ends and must be level with the top of the ties.

Ties are to be of redwood or cedar, 6x8 in. and 8 ft. long. Rails are to be 60-lb., Am. Soc. C. E. section, of bessemer steel, conforming to the standard specifications of the American Society for Testing Materials as revised in 1914. Tie plates are expected to be used on all ties. Ties are to be spaced 2,880 per mile. Two spikes are to be driven in each tie on the outside of the outer rail on all curves sharper than 8°.

Structures on the Line

Both ballast-deck and open-deck trestles are specified. The ballast rests on a layer of 3x12-in. flooring over the trestle stringers. This flooring is covered by ⅛ in. of approved asphalt roofing, reinforced by an extra asphalt coat of about ½ lb. per sq.ft.

A bridge of 220-ft. span is to be built across the Tuolumne River near Jacksonville, and with three plate-girder spans on the approaches will require about 600,000 lb. of steel. The bridge is to be designed for Cooper's E-35 load.

Eight redwood water tanks of 100,000 gal. each and two steel oil tanks 17x12 ft., to contain fuel oil for the locomotive, are required, as well as station buildings.

The contractor is to complete the entire railway ready for operation within 300 days from the signing of the contract and is to be subject to a penalty of $750 per day for each day's delay in completion over the contract time. No work on Sundays is permitted.

Concrete Pile and Cylinder Foundations at Charleston

SYNOPSIS—Special methods in securing foundations of reinforced-concrete torpedo-boat berth at Charleston, S. C., described in "Engineering News," Nov. 4, 1915. Piles spudded down and cylinder foundations laid in novel manner.

In the construction of the government torpedo-boat berth at Charleston, S. C., foundation difficulties arose because of the subsoil of marl and a tough but plastic clay overlying the marl in the outer half of the site.

The reinforced-concrete piles for the quay wall, 18 in. square and 50 ft. long for the three rows on the river side and 16 in. square by 40 to 45 ft. long for the other two rows, were made nearby on land, in horizontal

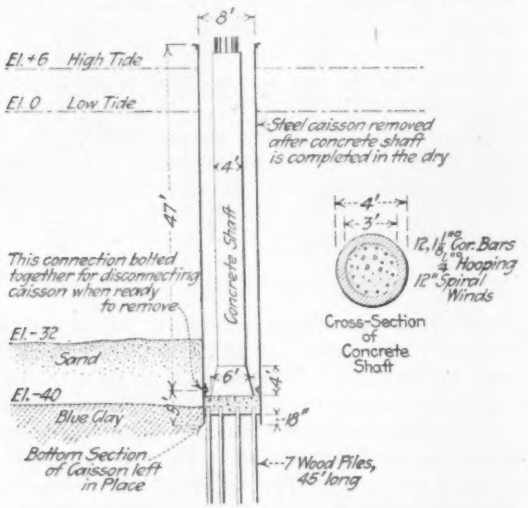


FIG. 1. CYLINDER, COFFER-DAM AND CONCRETE PILE

molds. The reinforcement consists of four longitudinal 1¼-in. square corrugated rods and ¼-in. ties 18 in. on centers. The piles were allowed to set 30 days before being moved. When ready for driving they were picked up by a 15-ton locomotive crane and loaded on a barge, which was towed to the point where the piles were to be driven.

Each pile had a 2-in. jetting pipe molded in it longitudinally for a distance of 17 ft. from the point, with two 30° bends at the upper end to bring the pipe to the face of the pile.

The piles penetrated approximately 10 ft. through soft mud, 15 ft. through stiff blue clay and 15 ft. into marl. Attempts to drive them with a hammer failed. It was impossible to get them down into the marl more than a short distance, even after jetting down four 2-in. holes in advance of the pile. In this difficulty a "spudding" method was adopted.

With the pile standing in the leads of the driving rig at the point where it was to be sunk, hose from a pressure pump was connected to the 2-in. pipe in the pile and the pump started. Then the pile was raised 2 ft. and allowed to drop, and this process was repeated in as rapid succession as possible until the pile was down within 6 in. of the proper depth. The water was then cut off and the last 6 in. spudded down in the same way, but without any water being forced through the jet pipe.

The derrick traveler shown in Fig. 2 was used for driving the piles. Guide-blocks bolted to the front of the forward cross-piece of the traveler frame furnished an upper guide for the piles. As a lower guide a cross-timber just above water level was supported from the traveler frame and braced to it, and sawed through in the way of each pile row so that it could move forward past the piles already driven. Swinging backing-strips were fastened to the timber behind the openings cut in the way of the pile rows, to serve as rear guides. The pile was lifted and dropped by the hoist rope of the derrick, which was fastened to a hook that was clamped to the four reinforcing rods of the piles that extended 18 in. beyond the head of the pile.

Piers Built in Cylinder Coffer-Dams

The concrete piers supporting the causeway and outer pier were built inside 8-ft. cylinder coffer-dams, 42 to 52 ft. long, made of ⅜-in. steel plate. The lower 5-ft. length of the coffer-dam was connected to the remainder by a butt joint fastened by bolts through angle rings riveted on the inside. The bolts were spaced 12 in. on centers, and for additional tightness the joint was calked with oakum. This cylinder was driven into the river bottom until it entered 4 ft. into the blue clay, and the inside was excavated to a short distance above the cutting edge. Then the foundation piles were driven in it, the bottom sealed with concrete, a form set up on the seal concrete and the shaft concrete poured. Finally the coffer-dam cylinder was removed, after unbolting the joint near the bottom. The 5-ft. cutting-edge section was left in the ground as part of the pier.

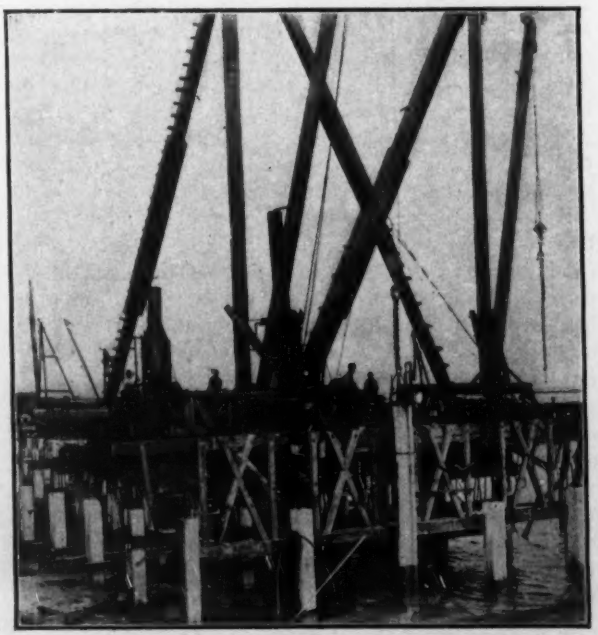


FIG. 2. DERRICK TRAVELER FOR PILEDRIVING

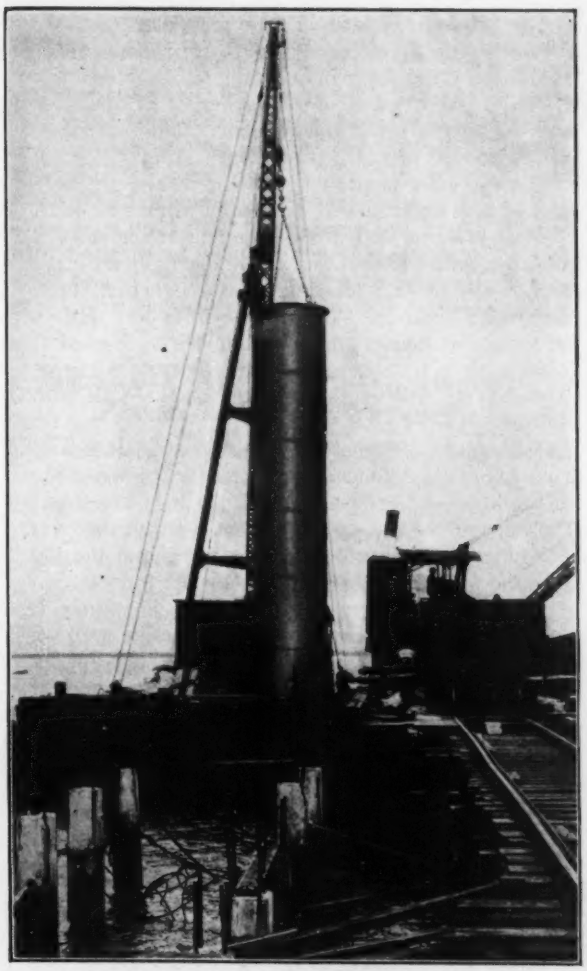


FIG. 3. LOWERING COFFER-DAM CYLINDER TO PLACE

Fig. 3 illustrates the procedure of setting the cylinders. This work was done at slack water, to avoid disturbances from the strong tidal currents. The cylinder, with bottom section attached, was picked up by a 25-ton floating derrick equipped with an 85-ft. steel boom, centered carefully while being held about 15 ft. above the bottom and allowed to drop so as to sink deep into the river mud. When thus placed the cylinder was braced three or more ways with light tackle to guide it during the subsequent driving. A grillage of 24-in. steel I-beams riveted together and with guides for a steam hammer attached was placed over the top, and a No. 1 Vulcan hammer with 5,000-lb. ram falling 42 in. held in these guides would drive the cylinder down to the required grade. The aim was to drive the cutting edge of the bottom section not less than 4 ft. into the clay, or 2 ft. into the marl, which amounts were generally sufficient to seal the bottom against blowing in when the cylinder was pumped dry.

A ¾-yd. orangepeel bucket, operated by the derrick scow, excavated the cylinder to the desired grade. In some cases the driving was resumed after excavation, to get the desired penetration of the cylinder for safety against blowing in of water. After excavating with the orangepeel, in the case of marl bottom the cylinder was pumped dry and the bottom leveled by hand digging, to receive a 3-ft. bottom course of concrete placed in the dry. Where the bottom was clay, the cylinder was pumped out, the bottom leveled by hand digging and the seven piles accurately spotted in the dry. If the clay was very firm, the piles were driven in the dry, but the cylinders were generally filled with water before piledriving was begun. After all the piles were driven, the cylinders were again pumped out, the piles sawed off and the 3-ft. bottom course of concrete placed in the dry.

Difficulties in Sinking Cylinders

The shoreward half of the causeway went through the old berthing pier, built of creosoted piles, and in placing the cylinders for this portion of the causeway the old piles interfered greatly. The ⅜-in. plate of the cylinder buckled easily at the cutting edge when it struck a pile, so that it was necessary to remove the piles before placing the cylinders.

The old piles were so badly eaten by borers and had been driven so deep into the clay that it was found necessary to use dynamite to remove them. For this purpose a 4-in. pipe with a 4-in. tee on top was connected to a 2½-in. fire hose supplied from a pressure pump, and the pipe was jetted down alongside the pile to be removed until its bottom was about 1 ft. below the proposed bottom of the cylinder. Then the water was shut off, the plug of the tee on top of the pipe removed and four or five sticks of 60% dynamite were lowered through the pipe until they rested on the bottom. The pipe was then raised about 10 ft. for protection, a good strain taken on the pile with the derrick and the dynamite exploded. This would break the pile off at the point of the explosion, and the strain from the derrick in connection with the lifting action of the dynamite was sufficient to pull the upper part of the pile. This method of using dynamite 45 ft. below water proved very successful.

Trouble was also caused by logs which had caught in the old piling and had become embedded in the river bottom. Many of these were 18 to 24 in. in diameter and were embedded as much as 15 ft. It was necessary to do considerable dredging with the orangepeel bucket and to shoot off blasts of dynamite under them before they could be removed with the bucket. Here also the dynamite was placed through the 4-in. pipe, as was done in removing piles.

Driving Foundation Piles in Cylinders

When it was decided to drive foundation piles for the concrete cylinder piers where they rested in clay a special floating piledriver was designed, as the piles had to be driven to a top elevation of from 36 to 50 ft. below high water. This driver was built on a 22x70-ft. barge, with the head end cut square; 68-ft. leads were built on the barge, overhanging the end of the barge 8 ft., with the bottom of the leads about 10 ft. above the water line. Extension leads 60 ft. long slid in the fixed leads. They could be lowered down to within 10 ft. of the bottom of one of the cylinders and still leave sufficient of the extension leads between the fixed leads to hold them securely in line. A No. 1 Vulcan hammer slid between the extension leads; to its lower end was fastened a hardwood follower 44 ft. long, with a special steel striking head on top and a 14-in. pipe sleeve at the lower end. Two short pieces of channel attached to the follower 10 ft. above its lower end slid over the extension leads and held it in place.

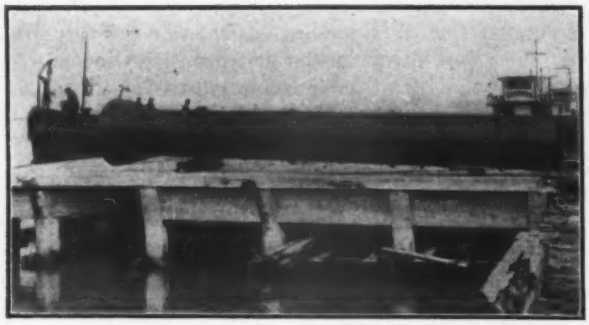


FIG. 4. EXPANDING STEEL FORM FOR CONCRETING PIER SHAFTS

The procedure in driving the foundation piles was to set them in place to form a ring 6 ft. in diameter and drive them in succession either to grade or to refusal. The piles were 45 ft. long, with 12-in. butt diameter, and had to be driven either so that the head would be embedded 6 in. to 30 in. in the 3-ft. bottom course of concrete or until the piles should take up with a penetration of ½ in. per blow.

The extension-lead driver rig worked very satisfactorily. In one cylinder all seven piles after being spotted were driven to grade in 1 hr. 20 min.

After driving and cutting off the piles the broken-up earth at the bottom of the cylinder was removed by hand digging in the dry, whereupon the 3-ft. bottom course of concrete, engaging the pile heads, was put in place.

The reinforcing cage of the pier shaft, comprising twelve 1⅛-in. square corrugated bars tied together with ¼-in. spiral hooping, of 12-in. pitch, was set directly after the sealing concrete was placed, so as to embed the rods at least 2 ft. in the concrete. This anchored the shaft concrete to the 3-ft. bottom course.

The day after pouring the footing course the top was cleaned of laitance or dirt and the form for the 4-ft. concrete shaft set in place around the reinforcing cage. This form, shown in Fig. 4, was a cylinder built up of $\frac{3}{16}$-in. steel, with conical enlargement at the bottom to form the spread foot of the shaft. The form was made with three vertical bolted joints. By unbolting one of these joints after the shaft concrete had set, the form would expand sufficiently so that it could be lifted off without difficulty.

Concrete for the footing course was run into a 1-yd. bucket from the chute of the concreting tower and the bucket lowered to the bottom of the hole and dumped. The concrete for the shaft, however, was run direct from the chute of the floating concrete plant into the top of the form and allowed to drop inside the form.

Great care was taken to have the form absolutely tight before placing and also to have the inside well oiled. On stripping, the piers were almost always perfectly smooth, an occasional stone pocket being immediately filled with mortar and floated with a wooden float.

Three days after the pier was poured the bolts in one of the vertical seams of the inside form were removed and the form lifted off with the derrick scow. The bolts

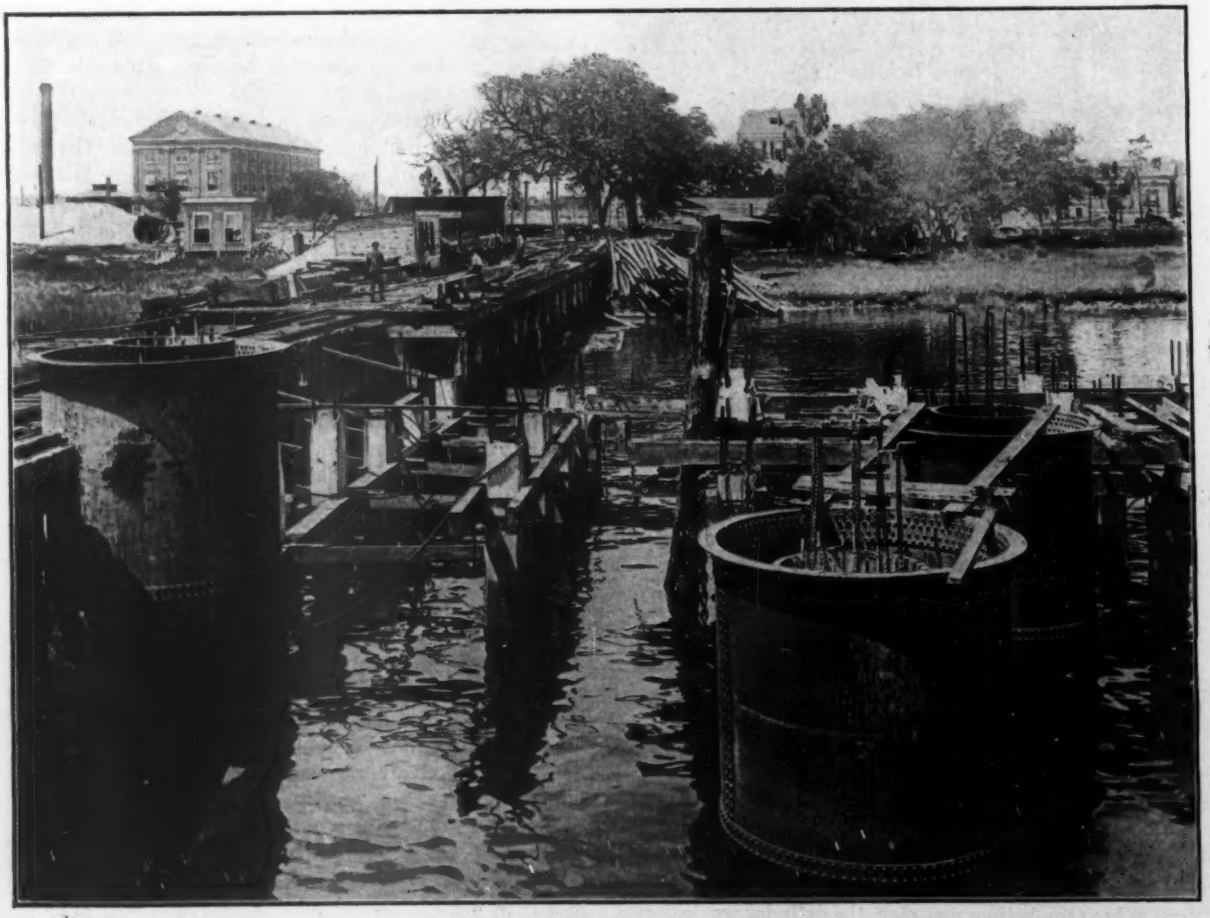


FIG. 5. PIER COFFER-DAMS OF CAUSEWAY

connecting the 8-ft. coffer-dam cylinder to its bottom section were also removed, and after the concrete pier shaft had been inspected the cylinder was hoisted away. However, as the specifications required that the concrete be protected from flowing water for at least 20 days, a square wooden box was lowered over the shaft to serve as a shield.

The work is being done by the Snare & Triest Co., New York City. G. C. Muñoz and G. P. Seeley were superintendents of construction and Captains F. T. Chambers and R. E. Bakenhus represented the Bureau of Yards and Docks, United States Navy Department.

Special Railway Track for Philadelphia Streets

The extensive waterfront improvements to be made at Philadelphia will require the laying of several miles of railway track along the streets to serve the docks and piers. For these lines a special design of rail section and track construction has been approved by the authorities of the city, the Pennsylvania R.R., Baltimore & Ohio R.R. and Philadelphia & Reading R.R. The following information has been secured from W. G. Coughlin, Engineer of Maintenanee of Way, Pennsylvania R.R.

The rail (Fig. 1) is of the grooved-girder type, 9 in. high and 5¾ in. wide, weighing 159 lb. per yd. and made in 33-ft. lengths. The guard side of the groove has a flare of 60° and its top is ⅜ in. below the railhead, although the paving is laid level with the railhead. On curves, however, the guard is carried up as high as the head and is made very heavy, as shown by the dotted lines. It will be observed that the guard is narrow and does not

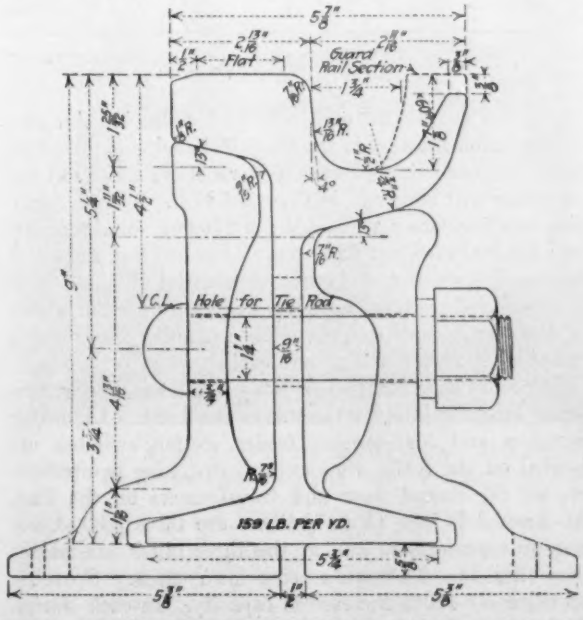


FIG. 1. 150-LB. GROOVED GIRDER RAIL FOR RAILWAYS IN PHILADELPHIA STREETS

have a horizontal outer flange to carry wagon wheels, as in some designs of grooved-girder rails for street railways.

The rail joints are of the Continuous make, with heavy splice bars of special design. The bars are only 26 in. long, with four 1¼-in. bolts spaced 5½ in. for the inner and 7½ in. for the outer bolts. The rails rest on ⅝-in. steel tie-plates 7x10¾ in., which have an outside top shoulder to hold the rail and a bottom rib to secure a hold in the tie. Ordinary ⅝-in. square spikes are used. For each tie-plate there are four spikes, only two of which hold the rail, the others serving to hold the plate tight upon the tie even if the rail spikes work loose. At the

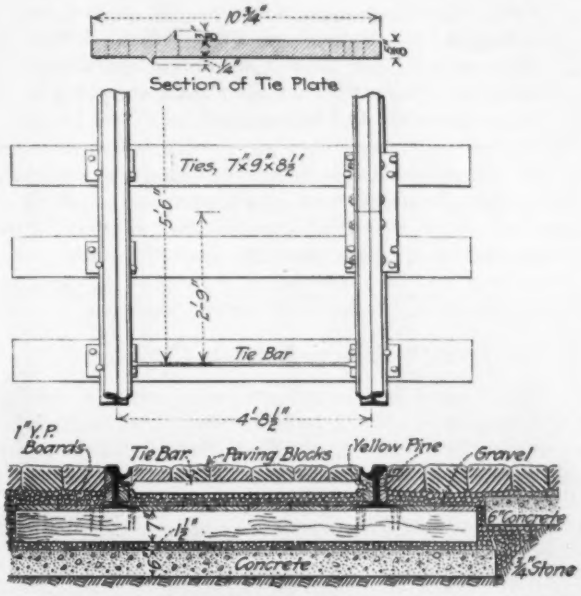


FIG. 2. RAILWAY-TRACK CONSTRUCTION ON PAVED STREETS IN PHILADELPHIA

joints, spikes are driven first in the slots of the splice bars, and after the bars have been drawn tight by the bolts extra spikes are driven through holes in the bars. The details of the trackwork are shown in Fig. 2.

The rails are laid with broken joints, and at intervals of 5 ft. 6 in. they are connected by flat tie bars 2¾x⅜ in., having 1⅛-in. threaded ends which pass through the rails and are secured by square nuts on both sides of the web. The ties are 7x9 in., 8½ ft. long, with 18 ties to a rail length.

The subgrade is 24⅛ in. below top of rail, and on this is a 6-in. concrete base. Stone ballast is laid with a depth of 1½ in. under the ties and is filled in between and around them. An unusual feature is the laying of 1-in. planks on top of the ties to support a 3-in. bed of gravel upon which the stone paving blocks are laid. In the spaces on each side of the rail web are laid yellow-pine fillers, 2x6 in. on the outside and 3x6 in. on the inside. Outside of the track the paving blocks rest on a thinner cushion bed on a 6-in. concrete base.

Home Garbage Disposal at Milwaukee—In every Milwaukee home there will be more or less fire from now on until the warm weather returns. These fires in stove and furnace should do more than give us warmth. There is little or no use for a garbage can in the Milwaukee home that is supplied with a good furnace. If garbage is drained, wrapped in paper and placed on the glowing coals of the furnace, it will be completely and effectively destroyed, and if this is done promptly after each meal and before there is decay there will be no disagreeable odors or nuisance of any kind. This method of destroying garbage has been tried by members of the Health Department for a number of years. None of them reports any trouble. To most of them the method has become a convenience so much greater than the garbage can that they would not think of returning to the can as a receptacle for the winter's accumulation of garbage.—Milwaukee "Health Bulletin."

Pennsylvania R.R. Electrifies Philadelphia District

SYNOPSIS—A 15 to 20% gain in capacity of each suburban passenger line can be secured by changing from steam to electric traction. Single phase selected for possible trunk-line operations. Overhead construction on the Paoli line; changes in signal system and rolling stock.

The Pennsylvania R.R. is now operating the suburban line between Broad St. Station, Philadelphia, and Paoli—the first of its electrified lines in that district. This change and the proposed electrification of the other suburban lines have been undertaken to increase the capacity of the Broad St. Station and relieve congestion.

Change Made to Increase Capacity

This terminal is of a stub-end type, with 16 station tracks approached by six main-line and three yard tracks on an elevated structure. At West Philadelphia the main lines branch off, going north to New York, west to Pittsburgh and south to Washington. The suburban service extends over six different routes, as shown in the sketch, Fig. 1.

All the plans for relief by physical enlargement involved extensive reconstruction and much delay, and therefore the possibilities of electric traction were analyzed. It was found that the change on the Paoli line alone would increase its capacity by about 19%, giving for the station as a whole an increase of about 8%. A similar increase in capacity would result from the electrification of the other suburban routes, and work on the Chestnut Hill line has already commenced.

Although the work done on the Paoli line includes the most expensive part of electrifying the whole district—the Broad St. terminal, with its elaborate yards and restricted property lines—yet there will be sufficient saving in operating cost over steam to pay the interest on the additional investment; that is, the increased capacity will be largely self-sustaining, which would not be the case with physical enlargement.

Alternating-Current System Selected

The 11,000-volt single-phase 25-cycle alternating-current system was selected, power being supplied directly to the train from an overhead catenary-trolley system. The primary factor affecting this choice was the possibility of long-distance operation over the entire suburban division, rather than the requirements of present short suburban electric service. This is in marked contrast to the reasons for the selection of 600-volt direct current for the New York terminal zone some years ago.

Although the possible future electric traction may extend over several railway divisions and cover various classes of service, yet the present installation is for suburban passenger service only and involves about 43 trains each way between Philadelphia and Paoli, west on the main line of the Philadelphia division. From Broad St. Station the main tracks are electrified for 20 mi. A coach yard at West Philadelphia and a coach and repair yard at Paoli have been equipped. A total trackage of 93 mi. is affected.

Electric Power Is Purchased

Power will not be generated by the railway company, but will be purchased from the Philadelphia Electric Co. For the present power will be taken from only one phase of the power company's three-phase generating system. It is planned to supply future electrification from the remaining phases. Current is generated by the electric company at its main station on the east bank of the Schuylkill River and is delivered over four three-conductor armored submarine cables to the railroad company's Arsenal Bridge substation on the westerly bank. Here the current is stepped up from 13,200 volts to 44,000 and is distributed over four single-phase overhead transmission

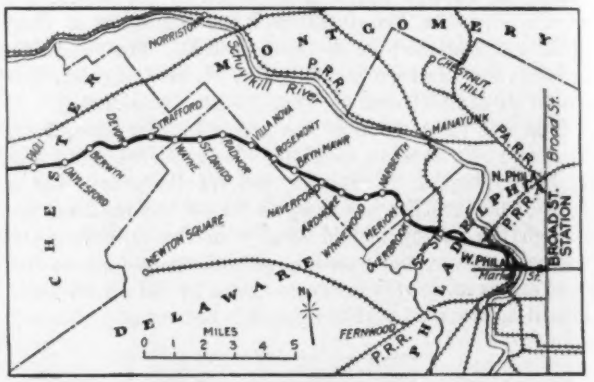


FIG. 1. MAP OF PENNSYLVANIA R.R. SUBURBAN LINES AT PHILADELPHIA

lines, two carried on each side of the track. All four transmission lines go to the West Philadelphia substation (Fig. 1); one pair continues to Bryn Mawr and Paoli and the other will later run to Chestnut Hill. The transmission-line insulators are tested for 250,000 volts puncture and 120,000 volts wet flashover. After erection the transmission lines were tested out at a potential of 66,000 volts to ground—three times working pressure (since the middle of the transformer secondaries is grounded the working potential is 22,000 volts above earth).

The substation equipment is housed in substantial fireproof brick buildings adjacent to the track. Lightning arresters and high-tension feeder section switches are located on the roof. Busbars and switching equipments are on the second floor and transformers on the first. At Arsenal Bridge (Fig. 1) there are three 5,000-kv.-a. step-up transformers and at the three other substations there are two 2,000-kv.-a. step-down units. Space is provided for 100% increase in capacity. In each station there are two small transformers stepping down to 440 and 220 volts for operating circuit-breakers for lighting and miscellaneous purposes. Sixty-cycle power is also provided in all substations to operate circuit-breakers in case of loss of 25-cycle traction power. There are no attendants in the substations, except at West Philadelphia. For the control of the other substations switchboards are provided in nearby signal towers.

The overhead-line construction in many ways resembles that worked out by the same engineers for the Norfolk & Western Ry., Elkhorn grade, as noted in *Engineering News* of June 24, 1915. The principal feature is the tubular-pole cross-catenary bridge supporting the longitudinal catenary and running wires. As shown in Fig. 2, a tubular steel pole is set in concrete foundations on either side of the track and double-guyed away from the track. Between the poles over the tracks are two cross-wires forming the cross-catenary bridge. Where there is no room for back-guying, self-supporting structural-steel posts have been used.

Details of Overhead System

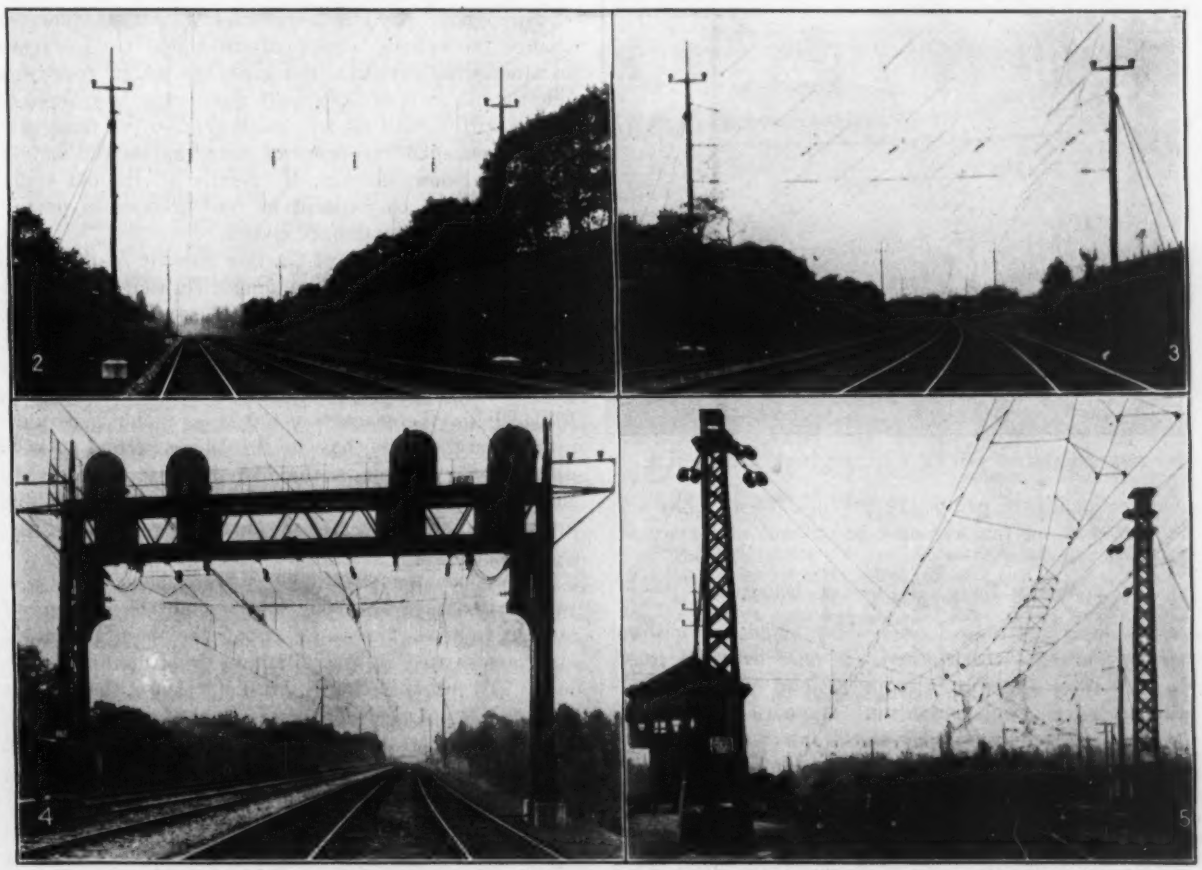
The tubular poles are built up of various lengths, sizes and weights of steel pipe welded together to meet the varying conditions of different locations. The guys are solid-steel rods, with a heavy turnbuckle near the ground end to permit of adjustment. The anchors are of the dead-weight type, consisting of a concrete slab reinforced with old rails. Where the guy rods pass up through the soil they are protected by steel pipe filled with grout. Each steel structure is grounded by connecting to a copper plate buried in coke.

The cross-catenary bridges are about 300 ft. apart on tangents but closer on curves, the spacing depending upon the degree of curvature. The insulators holding the longitudinal wires are over the center of each track, being offset on curves toward the outside.

The cross-wires are of galvanized-steel cable, the upper one usually ¾ in. and the lower one usually ½ in. in diameter. Both are socketed at each end with a turnbuckle at one side. The top and bottom wires are joined by vertical ¾-in. rods and malleable-iron clamps at the points where insulators supporting longitudinal wires are located. The insulators are of the three-disk suspension type. Each disk is 8 in. in diameter.

The main longitudinal messenger is a ½-in. seven-strand double-galvanized steel cable, strung with a sag of 5 ft. in a normal span of 300 ft. Every mile or two this cable is socketed and dead-ended on one of the structural signal bridges spaced about half a mile apart (Fig. 4). The messenger is insulated from the bridge by two or more sets of three-disk suspension insulators.

Every 15 ft. on curved track and 30 ft. on tangent track two wires are supported from the messenger. The top wire—called the auxiliary—is of No. 0 round copper, giving proper current-carrying capacity to the overhead system. The lower wire is a No. 000 grooved bronze alloy ("phono-electric"). Both wires are about 22 ft. above the top of rail, except where they pass under certain highway bridges



FIGS. 2 TO 5. OVERHEAD LINE CONSTRUCTION OF PENNSYLVANIA R.R. AT PHILADELPHIA
Fig. 2—On typical tangent. Fig. 3—On typical curve. Fig. 4—Signal and anchor bridge. Fig. 5—In Paoli yard

Provision Against Corrosion

In the terminal and along the first 5 mi. out, where the steam-locomotive traffic is dense and there is much smoke and corrosive gas, a noncorrodible tube hanger and hanger clips of monel metal and bronze are used. Where

the steam traffic is not dense wrought-iron strap hangers 1 in. wide by $\frac{3}{16}$ in. thick are employed. The main messenger cable is protected from corrosion at brass or monel-metal hanger clips by an inside zinc collar.

On tangents the hangers hold only the auxiliary messenger and the running wire is supported at mid-span. On curves where the two lower wires do not hang directly beneath the messenger both wires are supported at the same point from the hanger. Rigidity is here prevented by the chance for upward swing.

The tension in all cables is selected so that in hot weather there will be no sagging of the running wire, while in cold weather the contraction will not cause stresses beyond the elastic limit.

The running-wire catenary system for each of the four main tracks is separated electrically from the others,

FIG. 6. STEEL MOTOR CAR, PENNSYLVANIA R.R.

and sectionalizing points are provided at all crossovers so that parts of the line may be cut out of service for repair.

Wires Under Highway Bridges

The electrified route is crossed by numerous overhead highway bridges with limited clearance over the track. In such cases where it is impractical to raise the highway bridge, the trolley wires are dipped. The catenary bridges are so located that the highway bridges come in the center of the span and where the vertical height necessary to clear the catenary system is at a minimum. The overhead line is steadied by being held with post-type insulators on brackets on the bridge structure. Transmission lines on either side of the track are also carried down underneath the bridge.

To prevent curious pedestrians from coming in contact with the wires solid wooden fences, either vertical or inclined, and of sufficient height to shut out all view of the wires, have been erected. To protect trainmen general orders have been issued that no men be allowed on top of the cars in the electric zone.

Bonding Rails; Preventing Induction Interference

Each rail of the main-line track is double bonded throughout with No. 0 copper. Through interlockings only one rail of each track is bonded, but all the traction rails are connected together. The bonds have pin-expanded terminals, one being welded and the other soldered on, after the bond has been slipped under the fishplate. At each signal block the track rails are sectionalized by insulating joints. The traction current is led through impedance bonds connected around the insulated joints (allowing the 25-cycle traction current to flow, but preventing the 60-cycle signal current from passing).

Minimum inductive effect of traction currents on adjacent telephone and telegraph wires has been secured in much the same way as on the Norfolk & Western Ry. Series transformers are located about a mile apart along the line and the full traction current is led through the primary windings. The secondary coils are connected across insulated joints in the track so that at this point all returning traction current must pass through the rails. These transformers are shown in place on tower brackets of a signal and anchor bridge in Fig. 4.

Changes in Signal System

Throughout the electrified lines it was necessary to change the existing direct-current signal track circuits to alternating current. For about 15 mi. of four-track line the old form of semaphore signals has been replaced by one with hooded electric lamps arranged in three rows like the semaphore positions of clear, caution and danger. These are shown in Fig. 4. Nearly all the old signal bridges had to be replaced by heavier ones to provide anchorages for the catenary system.

The inductive effect of traction current in the signal circuits is killed by resonant shunts (capacity and resistance in series, adjusted so that they present zero impedance for the 25-cycle current).

Power for the operation of the signals is supplied by the West Philadelphia and Paoli substations. At West Philadelphia there are two 150-kv.-a. motor generators running on the trolley bus and furnishing current at 3,400 volts and 60 cycles over three single-phase feeders. An emergency transformer carried on an outside supply furnishes single-phase, 60-cycle current for signal lighting in emergency.

Throughout the electrified zone, telephone and telegraph wires have been put underground on account of damage from sleet storms. Along the main line these wires are carried in clay conduits or bituminized fiber ducts. To minimize induction disturbances the conduit is located as near the edge of the right-of-way and as far from the tracks as possible. The main signal-power cable is lead-sheathed and carried in a pitch-filled pump log run on top of the main bank of conduits.

Besides the usual telephone facilities between substations and between the power director and train dispatchers, there are permanent telephone boxes on every signal bridge—approximately half a mile apart.

Steel Rolling Stock

The traffic is carried entirely by motor cars. The standard steel suburban coaches have been used without structural changes. This was possible, since the requirements for mounting electric apparatus were considered at the time when the car was introduced. Each car carries two 225-hp. single-phase air-cooled motors, mounted on one truck. The other truck does not drive. Each motor car is complete with one 11,000-volt pantograph trolley, oil circuit-breaker, main transformer, switch group and limit switch, resistance, and two master controllers, one motor-generator set for supplying the control energy, one

control battery, four nine-point train-line receptacles and one train line jumper, one combined blower and compressor unit, and one complete air-brake equipment. All of the main pieces of electrical apparatus are mounted on one end of the car and the air-brake equipment is mounted at the other end. This puts approximately 60% of the total car weight on the driving wheels. The cars are fitted for double-end control and operation.

The motors are connected in series and are started and operated up to about 15 mi. per hr. as repulsion motors with the auxiliary field, armature and main field in series. The running steps for higher speeds place the auxiliary field on one part of the car transformer and the armature and main field on the other portion. Subsequent steps are obtained by increasing the motor voltages. Between the motor shaft and the driving wheels are flexible gears, each made up of a rim, a center and intermediate springs, which equalize the rapidly pulsating torque of the single-phase motor and prevent unpleasant vibration in the car frame.

The air-brake equipment is electro-pneumatic (the Westinghouse U. C., which has been generally adopted by the Pennsylvania R.R.). The application is made simultaneously on all the cars by electrically controlled release, service and emergency sections of a "universal valve" which replaces the "triple valve" of the older air-brake systems. This equipment may be used with either steam or electric traction.

Engineering and Construction

The design and construction described were carried out by Gibbs & Hill, of New York City, as consulting engineers for the railroad company, but in coöperation with the engineering department and officials. All construction except for substation buildings and inspection buildings (which were built under outside contracts) was carried on by specially organized force. The motor-car equipment and important electrical apparatus was furnished by the Westinghouse Electric and Manufacturing Co., but the mounting of the car equipment was carried out by railroad forces under the motive-power department. The signal equipment and changes in telephone and telegraph lines were designed and installed under the direction of the signal and telegraph departments, the equipment being made by the Union Switch and Signal Co. Tubular poles were made by the National Tube Co., structural material by the McClintic-Marshall Co. and Belmont Iron Works; overhead wires by J. E. Roebling's Sons Co., Waclark Wire Co. and Bridgeport Brass Co.

⌘

Flat or Adjustable Bearings in Brick Tests—Ordinary flat (rigid) bearings for testing brick in compression are nearly as good as self-adjusting bearings, according to an extended series of tests just reported in the October "Journal" of the Association of Engineering Societies. E. L. Baker and A. F. Suss, students in Washington University, made the tests. The series included about 300 tests. To get comparative values, each brick was first broken in cross-bending and then the two pieces tested in crushing, each by a different method. The bearing surfaces were chipped flat with cold-chisel and hammer, but were not bedded. Two forms of spherical bearing were used—(1) a one-piece bearing, consisting of a rocker shoe the curved surface of which rested on the platen of the testing-machine, while the other (flat) face supported the brick; and (2) a ball-and-socket bearing. The comparison values of crushing strength obtained in the tests are: One-piece spherical bearing to flat-plate bearing, 1.03; ball-and-socket bearing to one-piece bearing, 1.12. The ball-and-socket bearing gave the highest average crushing strength, 9,340 lb. per sq.in. These brick averaged 2,600 lb. per sq.in. in modulus of rupture.

Load Distribution on Concrete Slab Floors

The Highway Department of the State of Ohio (Clinton Cowen, Commissioner) has issued as Bulletin No. 28 a résumé of a long series of tests made to determine the distribution of concentrated loads to the beams and girders under a reinforced-concrete floor slab. The tests were both on laboratory specimens and on the floor of an actual bridge. The pamphlet is too long to abstract here, but as an indication of its contents the summary of the results is given below. The object of the tests was to obtain, if possible, a sufficient knowledge of the distribution of loads through and by concrete floor slabs to enable the designer rationally to proportion the joists of a slab floor, and also the slab itself to carry concentrated loads.

Practically all heavy loads on highway-bridge floors except the dead-load are concentrated on small areas, such as the wheels of a road roller or heavy motor truck. In the design of reinforced-concrete floors, therefore, there are two assumptions which must be made:

(1) In designing the supporting joists or stringers, how much of a concentrated load on the slab is carried by the joist immediately below it and how much is distributed by the slab to other joists? (2) In designing the floor slab itself, what width of slab may be considered as carrying a concentrated load resting upon it?

A satisfactory theoretical solution of these questions is impossible, as it involves the elastic properties of both concrete and steel in flat slabs and beams with multiple supports, and requires so many assumptions that any results thus obtained would be little better than assuming the distribution at once.

The following conclusions regarding the distribution of concentrated loads on a reinforced-concrete slab to the floor joists seem to be warranted by these tests:

1. The percentage of reinforcement has little or no effect upon the load distribution to the joists so long as safe loads on the slab are not exceeded.

2. The amount of load distributed by the slab to other joists than the one immediately under the load increases with the thickness of the slab.

3. The outside joists should be designed for the same total live-load as the intermediate joists.

4. The axle load of a truck may be considered as distributed uniformly over 12 ft. in width of roadway.

5. If the slab has ample grip on the upper flange of the I-beam and is continuous over the floor-beams, and the joists are riveted to the web of the floor-beams, the live-load stress in the joist may be but one-half as great as for a similar load on the bare I-beam supported at its ends.

6. Under these favorable conditions the axle load in a panel of not more than 20 ft. may be assumed as uniformly distributed over two-thirds of the length of the joists considered as simple I-beams supported at the ends. Without these conditions the load may be assumed as uniformly distributed over a length of at least 5 ft.

In a slab of a certain span and indefinite width there is some width symmetrical with the load beyond which a single concentrated load will have no effect. The stresses in this slab will be a maximum under the load and will decrease in each direction from it.

The "effective width" of a slab is that width used in designing over which a single concentrated load may be considered as uniformly distributed on a line down the middle of the slab parallel to the supports.

The tests of slabs seem to warrant the following conclusions:

1. The "effective width" is affected very little by the percentage of transverse reinforcement (parallel to supports).

2. The "effective width" decreases somewhat as the load increases.

3. The "effective width" in percentage of the span decreases as the span increases.

4. The following formula will give a safe value of "effective width" where the total width of slab is greater than $1\frac{1}{2}S + 4$ ft.: $e = 0.6S + 1.7$ ft., where e is effective width in feet and S is the span in feet.

Tests and studies of the same problem are being made by Philadelphia, but so far without results.

Substructure of the Lake St. Bascule Bridge at Chicago

By Hugh E. Young*

SYNOPSIS—Foundation work on one of the heaviest drawbridges ever built, a double-deck double-leaf bascule. Anchor piers in open wells. Steel sheetpile coffer-dam for main piers.

The double-deck double-leaf bascule bridge now being built across the river at Lake St., Chicago, was described by the writer in *Engineering News*, Nov. 4, 1915. The substructure consists of a tailpit and an abutment at each side of the river. The tailpit is a concrete box resting in four cylinder piers carried down to solid rock, the walls of the pit acting as beams. The piers are spaced 34 ft. laterally so as to provide for a possible future double-track subway on Lake St. North of the tailpit are foundation walls for the operators' houses. These rest on pile foundations. The abutments are supported on groups of piles driven on each side of the street, and act as beams, spanning the space that would be occupied by a future subway.

The depth of excavation for the tailpit necessitated special care to protect adjacent buildings and the elevated railway structure. Work did not proceed simultaneously on both sides of the river due to the fact that the scows for delivering material and removing excavated material effectually blocked the channel adjacent to the pier under construction, and it was necessary that one draw be left clear for the passage of the river traffic. Work was started Mar. 30, 1914, by the Fitzsimons & Connell Co., contractors for the subway.

General Method of Working

The general method of conducting the work on the east side was as follows: Excavation was carried down to a general elevation of about 3 ft. above datum, giving about 38 to 40 ft. clear headroom under the elevated structure. A land piledriver was rigged up to clear the elevated structure, and all piles were driven. It was occasionally necessary to dig a hole for the point of the pile, in order to get the butt in under the hammer. By doing this piles about 25 ft. long could be driven.

As the four-story brick building on the north side apparently rested on floating foundation, steel sheeting was driven to protect it. The contractor braced the structure by tying the outside walls together by means of rods connected to timbers on the outside faces. During the excavation a reinforced-concrete wall inclosing the subsidewalk basement of the building on the south side of the street had to be removed. This was broken up by light charges of dynamite without any accident. The foundation piles under the operator's house on the north side of the street were also driven.

The coffer-dam was next constructed. This consisted of a double row of Lackawanna steel sheet-piling for the portion in the river and a single row of steel or wood piling for the portion on land. Where channel clearances permitted, two rows of sheeting were driven about 7 ft. apart; but the rows for the front wall of the coffer-dam were 3 to 5 ft. apart. The two rows were tied together with rods and the space between filled and puddled with clay dredged from the river. The sewer on the north side was diverted through a trough outside of the coffer-dam.

A derrick was then erected on the north side to handle the excavation from the coffer-dam. Its mast was carried by a group of piles near the north support of the rear girders supporting the elevated structure, and the stiff-legs were anchored to the coffer-dam. The derrick hoist was erected on a pile foundation outside the coffer-dam.

As the excavation for the tailpit proceeded the coffer-dam was braced by four horizontal tiers of 12x12-in.

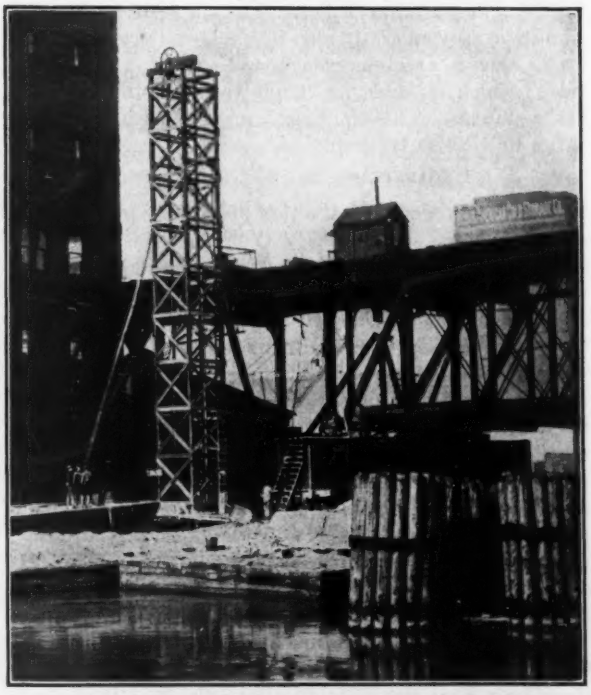


FIG. 1. CONCRETING PLANT AND TOWER

The materials are delivered on scows. The view shows the west end of the old swingbridge, with the columns of the elevated railway approach carried temporarily on a box girder spanning the tail-pit of the new bridge

cross-bracing. One tier of bracing was placed about 18 in. above datum; the others were 6, 13 and 19 ft. below datum. The bracing was supported by means of old piles and partly by suspension rods from the temporary box girders carrying the columns of the elevated approach. The material was loaded into buckets, which were lifted out by the derrick and dumped into a scow alongside the coffer-dam. The excavation was rushed day and night, with men working in three shifts.

When the excavation for the tailpit was practically completed, that for the subpier wells was started. The 8-ft. anchor-pier wells were started first and were dug as open wells, being excavated about 5 ft. at a time and then lined with 3x6-in. tongued-and-grooved maple lagging in

*Bridge Designing Engineer, Department of Public Works (Bureau of Engineering), Chicago.

lengths of 4 ft. 8 in., each length being held in place with two or three rings of 1x4-in. steel. These rings were made in two sections, with a lug bent on the end of each for bolting together. The wells were excavated in this manner to solid limestone rock, the bottom of the well extending 1 ft. into the rock to insure a good bearing.

The material encountered in these wells was blue clay from 20 ft. (where the well started) to about 60 ft. below datum. This ranged from soft clay at the top to very stiff clay at the bottom. From 60 to 75 ft. below datum the material was a mixture of tough blue clay with gravel and boulders; from 75 to 90 ft., a mixture of soft clay and sand; from 95 to 105 ft., gravel and sand. At 105 ft. a water-bearing stratum of sand was encountered, extending to rock at 107 ft. below datum. The quantity of the water was not enough to necessitate an air-lock, the water being removed by a centrifugal pump; but progress was much retarded. It took from Aug. 14 to 28 to excavate these two wells.

Concreting Plant Erected

While these wells were being put down the concreting plant was erected. The mixer plant was placed alongside the dock of the building at the south side of the street, on a pile foundation. In front of the mixer was erected a tower, similar to that shown in Fig. 1 (which is the tower at south side of west coffer-dam), for elevating the concrete to metal chutes that distributed the concrete to the forms. Handling concrete in barrows was unnecessary except for minor parts of the work beyond the reach of the chutes. These chutes were supported from the elevated structure by means of tackle and blocks, and were easily adjusted. Material was delivered to the river plant on scows, being wheeled from the scows in barrows to the mixer hopper.

When the excavation of the wells was completed the concrete filling was started. The concrete was mixed wet and dropped through pipes into the well. The work was inspected at regular intervals during the pouring of the concrete to see that it was being properly filled around the reinforcement and that too much water was not accumulating on top. As the concreting proceeded, the lagging and rings were removed, except for the bottom 15 to 20 ft. of the well. The concreting of the anchor-pier wells was completed on Aug. 31.

Excavation Started

The excavation for the 12-ft. wells for the river pier was started Aug. 31, as soon as the anchor-pier wells were concreted, and proceeded uninterruptedly until Sept. 10, the wells having been carried down to 95.5 ft. below datum. Work was then suspended temporarily, as the contractor encountered some difficulty in getting proper sand for concrete, and decided not to finish the well to rock until the proper materials were available. After a short delay excavation was again started, and rock was reached Sept. 15. Fig. 2 shows the rings and lagging in place, the line of electric lights in the well to illuminate it, and the pump for keeping the well dry.

Concreting was started immediately after the excavation was completed and concrete brought up to 93 ft. below datum, when work was again suspended due to the rejection of the sand by the inspector. This was lake sand from the eastern shore of Lake Michigan, and was finer than that called for in the specifications. After a delay of about a day suitable sand was procured and delivered in cars to a dock a short distance from the bridge, from which place it was brought to the site on scows. The work was then started again, the wells being concreted to the top. The lagging in these wells was left in place from the bottom up to 90.4 and 91.6 ft. below datum for the north and south wells respectively. The wells were completed on Sept. 21.

A bad leak in the south wall of the coffer-dam was noticed on Sept. 14; and it required considerable pumping to keep the water low enough to continue to work. The water entered at the connection of the double row of steel sheeting with the dock, and then through the wood sheeting which forms the part of the coffer-dam on the land. Considerable material was dumped in the river along the

FIG. 2. INTERIOR OF SHAFT FOR ONE OF THE FOUNDATION PIERS

The shaft is 12 ft. in diameter and 107 ft. deep, extending 1 ft. into solid rock. The pump and the electric lights are shown. The shaft is ready to be filled with concrete

dock where the leak started, and finally stopped the leak, on Sept. 24.

Forms for the tailpit proper were prepared at the time the piers were being constructed, and as soon as these piers were finished (Sept. 29) the concreting of the tailpit was started. The floor of the pit, a 5-ft. slab, 62x64 ft., was finished Oct. 3. The construction of the floor requires all piles and other supports for the bracing of the coffer-dam to be cut off above the top of floors, in order to avoid holes in the slab. The bracing is made to hold itself by trussing. The side walls and the river and anchor piers (forming the end walls) of the pit were then brought up to an elevation 1 ft. below datum, and were completed on Oct. 22. The overhanging parts of the side walls were temporarily omitted, so as to clear the piles

under the girders supporting the elevated structure. The outside walls for the operator's house were also brought up to the bottom of the cross walls joining them to the tailpit.

In order to make the pits watertight, a 6-in. layer of cement mortar, made of 1 part portland cement and 2 parts fine aggregate, was placed in the floors and on the outside faces of the walls, up to above datum. The mortar facing on the walls was applied by means of a mortar board, used as a form, which was placed inside the wall forms at the proper distance to give the required thickness of facing, the board being raised as the facing course was carried up with the concrete. The mortar was made in a mechanical mixer, and was delivered in chutes to a box, from which it was carried to the forms in coal scuttles or pails. The concrete was delivered by chutes directly into the forms. From the time the coffer-dam was first pumped out until the concreting was finished the work was carried on continuously, the men working in three shifts, except on Sunday (midnight to midnight), when no work was done.

East and West Abutments

At this stage of the work excavation was started for the west abutment, and the excavation for the east abutment was finished and the forms were constructed. After the concrete in the tailpit had set the girders supporting the elevated structure were shored up on the masonry, as described in the previous article (and shown at *A* in section *CC* of Fig. 3). The piles were cut off so that the overhanging walls and top of the walls under the operator's house could be finished. This work was finished on Nov. 12. The concreting of the east abutment was started Nov. 13 and finished Nov. 25. The concrete plant was then moved to the dock on the west side of the river (Fig. 1). Foundation piles for the west abutment were then driven in the same manner and under the same conditions as those for the east abutment.

The forms for the east tailpit being removed, the space between the coffer-dam and tailpit was filled with water. Under full head of water the pit showed a small leak in the north wall at an elevation of about 1 ft. below datum at the junction of the side wall and overhang. This stopped as soon as silt filtered into the crack, and the pit was then accepted by the city. The coffer-dam was then removed and the channel dredged of all excess material, such as puddling and excavation.

At this same time the west coffer-dam was being constructed and foundation piles for the operator's house were being driven. Excavation was also made for the new portion of the 5-ft. sewer on the west side, and the sewer constructed up to the abutment, where a temporary bulkhead was placed and an outfall constructed under a freight house into the river.

Work on the West Pier

Pumping out of the west coffer-dam started Jan. 11, and as the excavation proceeded the dam was braced as described for the east dam. Excavation for the foundation wells under the west anchor pier was started Jan. 23 and for the river wells Jan. 25. On Jan. 31, due to a rise in the river, the water overflowed the puddle wall of the coffer-dam and partly filled the wells, which were then pumped out. Excavation in the anchor-pier wells was done after the river-pier wells were finished. Water was encountered in the wells at 101 ft. The excavation for the river-pier wells was completed Feb. 7, having been carried to a depth of 109 ft. below datum and 1 ft. into solid rock. Concreting was finished in these wells on Feb. 12, the lagging below 94 ft. and 87.25 ft. below datum being left in place.

The excavation for the north anchor-pier well was completed on Feb. 10 and the south well Feb. 11. They were immediately concreted, work being finished Feb. 15. The lagging was left in place below 82 ft. and 80 ft. below datum for these walls. The tailpit forms were then constructed and concreting commenced Feb. 16. The floor was finished Feb. 20, and by Mar. 6 the walls were brought up to about 1 ft. below datum.

On Mar. 5 a leak developed in the south wall of the coffer-dam adjacent to the building, and this flooded the coffer-dam to river level. This break was repaired and the dam pumped out again.

During the time that excavation for the operator's houses was proceeding the forms for the west abutment were erected and the wall was concreted. This work was started Mar. 12 and finished Mar. 19. The box-girder supports of the elevated structure were then blocked up on the masonry and the overhanging walls and operator's house walls concreted, finishing Mar. 29. As a precaution against freezing the finished work was covered with tarpaulins.

The test of the west pit was made Mar. 31, and the coffer-dam was removed Apr. 2. The plant was then removed, docks were repaired where they had been cut for the coffer-dam, backfilling was placed, the permanent pier protection constructed, and the site cleaned up. The substructure contract was completed Apr. 28, or about 13 months after starting work. The girders supporting the elevated structure on the west side were left in place.

Activities at a War Port

Engineers on this side of the ocean who are concerned in the development of port works in an orderly, and sometimes in a dilatory, fashion will be interested in the feverish activity forced upon the port authorities of Archangel, Russia, in order to make that port equal to the greatly increased trade of the war. United States Commercial Attaché H. D. Baker reports upon this activity as follows:

Information has been received of continued improvement in conditions at Archangel since the time of my visit there early in August. Notwithstanding increasing arrivals of steamers, the goods are shipped out now within a reasonable time after they arrive, and considerable cotton and other merchandise that had accumulated there early this year have now found their way to interior destinations.

It is reported that very good progress has been made during the last month in railway improvements between Vologda and Archangel. It is possible now to run broad-gage cars from any part of Russia direct to Archangel, and for half the distance from Vologda to Archangel the line is double-tracked. The narrow-gage line will be retained, so that it will still be possible to make use of the narrow-gage equipment. Rapid progress is also being made on the line through Lapland between Kandalaksha and Kola. It is expected that it will be possible to run cars over this line after Jan. 1, 1916, so that Kola during the winter season can be substituted for Archangel as a port, there being no heavy ice there as there is at Archangel.

Three cargo ice breakers, one of 2,500-ton capacity and two of 500-ton capacity, will be put on the run for the winter season between Kandalaksha and Sorotskoe, the voyage probably taking about 24 hr. each way. Owing to their limited capacity, the ice breakers will probably be reserved for freight urgently required by the government service.

Erie Flood-Protection Report

One of the highest flood-flow figures on record is given for Mill Creek in a report on the Erie flood of Aug. 3, 1915, by Farley Gannett, Consulting Engineer. He reports to the city authorities that the maximum flow was about 1,000 cu.ft. per sec. per square mile of drainage area. As noted briefly in the news columns of *Engineering News* of Oct. 7, p. 718, he recommends dry-reservoir control to a maximum flow of 6,000 cu.ft. per sec. and channel improvement within the city by concrete lining and conduit.

Magnitude of Flood

The Mill Creek drainage area has been surveyed and found to measure 12.9 sq.mi. (instead of about 17, as stated in *Engineering News* of Aug. 12, 1915). Of the total, 2 sq.mi. lies within the city limits and is largely built over. The drainage area is hilly; the soil is gravel underlain by a gray slate formation, a combination likely to produce a high runoff.

The total fall of the stream from its source is about 500 to 730 ft. above Lake Erie. Within the city its average slope is 1.20 ft. and just above the city for 2 mi. the slope is 1.04 ft. per 100.

The rainfall in the city of Erie on Aug. 3 totaled 5.77 in. within 14 hr., including three separate spells about an hour apart in each of which about 1¼ in. fell in half an hour. It was suspected that higher rainfall occurred in the hill regions south of the city, where the sources of Mill Creek are located. Accidental collection of the rainfall in buckets and other containers, as a careful search disclosed, indicated precipitations of from 7 to 9 in. Mr. Gannett concludes that in the hills the rainfall was about 40% greater than at Erie. It was also more concentrated, starting later and ceasing more abruptly and earlier.

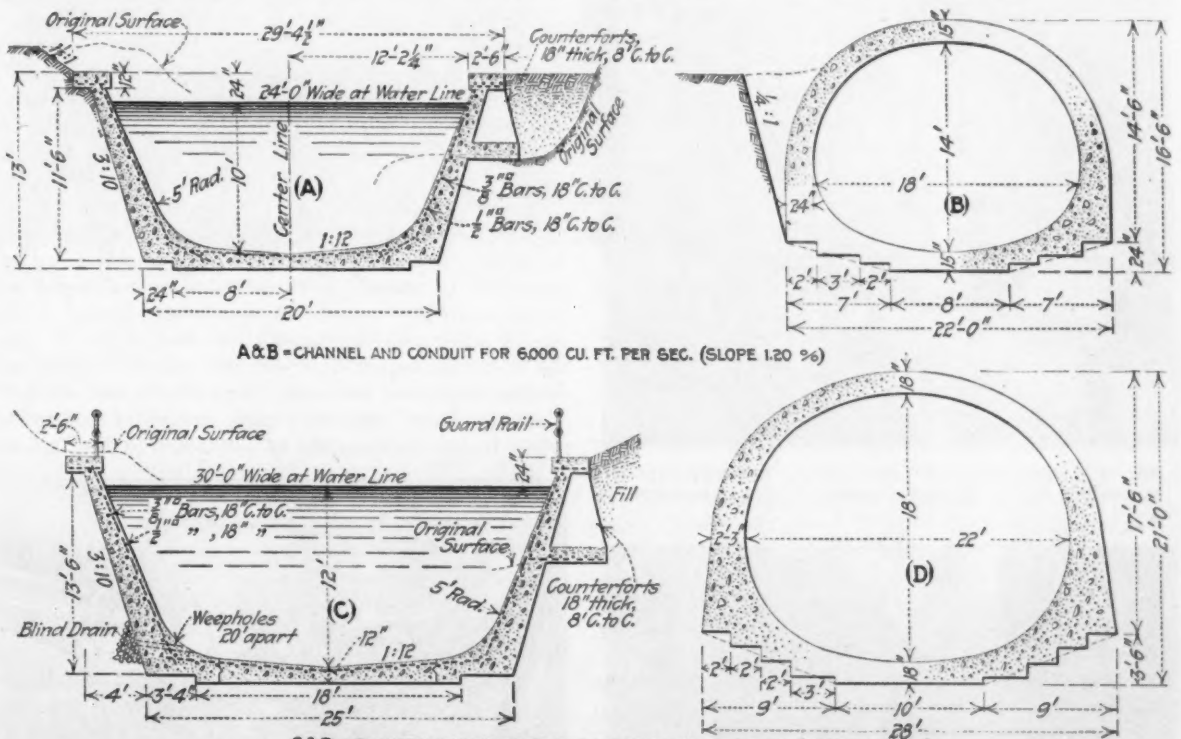
The time required for water to flow from the headwaters of Mill Creek down to the city line ranges from 1 hr. to 2 hr., the shorter period prevailing for the higher flood stages. This means that in the storm of Aug. 3 the water had a favorable opportunity to concentrate, since the successive periods of heavy precipitation were about an hour apart. The flow at 26th St., near the city limits, is estimated at about 11,000 cu.ft. per sec. The drainage area above this point is about 11.1 sq.mi. Mr. Gannett states that this is about nine times the extreme-dry-weather flow of the Ohio River below Pittsburgh, draining 23,000 sq.mi

Greater Flood Possible at Erie

By plotting the precipitations which have occurred at Erie in the last fifteen years and drawing a curve showing the variation of precipitation rates with duration, Mr. Gannett suggests that a rainstorm about 10% heavier than that of Aug. 3 is possible in Erie, even if no broader range of experience is depended on that the last fifteen years' records. This means that a more destructive flood may occur unless protection works are built.

The maximum flood flow assumed for the design of protection work is 12,000 cu.ft. per sec., and the channel-improvement project without storage is designed to carry this flow. In the project with storage the reservoir is designed to pass a flow of 6,000 cu.ft. per sec. and the channel is designed for this quantity.

The channel and conduit designs worked out for the course of the creek through the city are shown in the accompanying drawing, which shows sections to take care of the two flows noted.



CHANNELS AND CONDUITS FOR ERIE FLOOD PROTECTION AS SUGGESTED IN THE GANNETT REPORT

Field and Office

Lining Two 5-Ft. Tunnels by Use of Concrete Atomizer

At Rupert, Penn., the Delaware, Lackwanna & Western R.R. has recently completed the lining of two 5-ft. water-power tunnels with concrete sprayed into place by the "concrete atomizer," a machine controlled by Harold P. Brown, 120 Liberty St., New York City, in which a fluid concrete is blown through a nozzle by the force of superheated steam. The machine was described in *Engineering News* of Oct. 14, 1914.

FIG. 1. INTERIOR OF HYDRAULIC TUNNEL NEAR RUPERT, PENN., BEFORE LINING WITH CONCRETE

FIG. 2. STEAM BOILER AND CONCRETE ATOMIZER SET UP ABOVE TUNNEL

At Rupert a large flour mill built about 100 yr. ago is supplied with water for power by a pair of masonry tunnels about 5 ft. high passing under the bed of an abandoned canal for a distance of 150 ft. The mortar between the stones had been washed out, leaving a rough, imperfect surface, as shown in Fig. 1, and through these crevices a large leakage found its way to the tailrace. The railroad had purchased the canal many years ago and is under obligation to make all necessary repairs. The rough character of the stonework made troweling of little value and there was not sufficient headroom to permit the use of forms and poured concrete. Furthermore, the location of the tunnel so far below the bottom of the canal made it too expensive a matter to open the trench and repair the masonry from above. The satisfactory work of the atomizer last year on the railroad's Hoboken terminal and in the open cut at Newark, N. J., led to the use of this apparatus for repairing the tunnels.

Since the tunnels were too far from a railroad track to permit the use of steam from a locomotive, as had been done in previous work, a 20-hp. vertical boiler was set up above the tunnel. A superheater located in the bonnet and accessible through the stack (Fig. 2) gave the steam the superheat which the inventor claims produces a waterproof concrete of high density and strength.

It was feared that the small section of the tunnel might make it impossible to use steam in the inclosed

FIG. 3. CONCRETE ATOMIZER WITH DELIVERY PIPE LEADING DOWN INTO TUNNEL

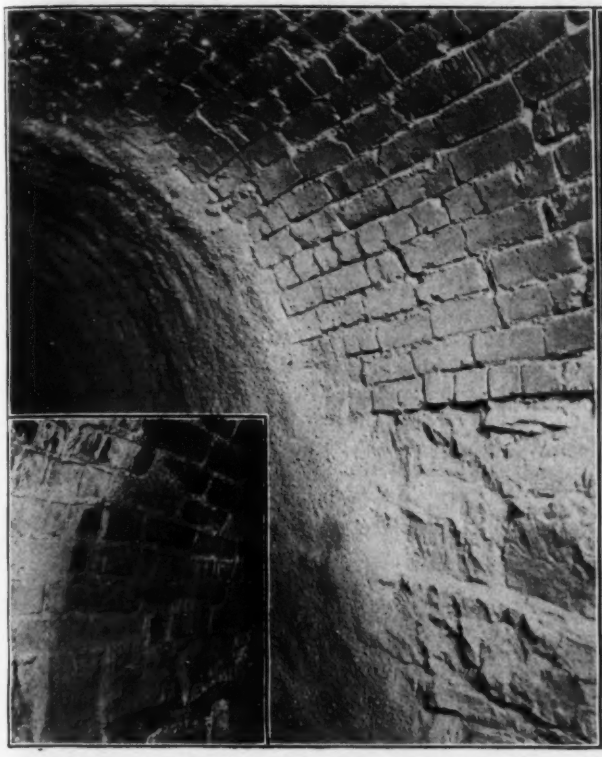


FIG. 1. ORIGINAL CONDITION

FIG. 2. WITH JOINTS CLEANED

FIG. 3. CEMENT-GUN FINISH AND TROWELED SURFACE

space without forced ventilation and that the exhaust steam and the high temperature would interfere with the work, but no trouble of this kind was encountered.

The stonework was first cleaned by a jet of steam and hot water from the atomizer. This was at once followed by a jetted stream of concrete, which adhered to the sides of the roof of the tunnel and filled all the crevices. For coarse aggregate very small pebbles were used, while sand was used for fine aggregate. The pebbles hammered the concrete into the cracks and crevices so that no troweling was necessary. While the surface formed is not as smooth as is desirable on some work, it is sufficiently smooth for the case at hand. In many places large stone had dropped from the roof and 6 to 8 in. of concrete was required to make the surface level.

The 20-hp. boiler proved to be ample for the work and the proper amount of water in the concrete was easily maintained. No difficulty was found in carrying the concrete several hundred feet from the mixer, and there was but little wear on the hose, probably because the wet cement acts as a lubricant.

Cement Gun for Relining Old Brick and Ashlar Sewers

By L. Chivvis*

The engineers of the City of St. Louis, Mo., have been experimenting to determine the availability of the cement gun for relining old sewers of large diameter, with the intention of purchasing an outfit for municipal use if the results obtained warrant the expense. Theoretically the capacity of rough brick and ashlar sewers could be sufficiently increased by a smooth mortar lining (smooth enough to justify a coefficient n in Kutter's formula of 0.013) to make possible a large saving in the cost of proposed relief sewers. Moreover, the sewers are improved by filling the joints in the old masonry.

Although computations were conclusive, there were several points on which no data could be obtained. For instance, it was not known whether this thin mortar coat would adhere to wet, slimy brick without previous treatment; and if treatment proved necessary, whether a sandblast or a wire-brush abrasive was the best method. Again, since smoothness of surface was the principal object, it was thought advisable to give the gunite a trowel finish, even if the natural film surface was destroyed and the waterproofing qualities impaired. Since the cement gun hitherto has been mainly used to give a weather or waterproof surface, this troweling was objected to by all the companies consulted.

A trial contract was entered into with a Chicago company for the use of a gun, a compressor and two operators. A length of sewer in which were combined both ashlar and brick masonry, and which was small enough so that all parts could be reached without the use of scaffolding, was selected for the experiment. Fig. 1 is a fair representation of the original surface.

In preparing the sewer for the mortar coat, wire brushes were first tried and found wanting. The sandblast failed because of insufficient air. The method finally adopted was to pick and scrape the mortar out of the joints with a tuck-pointer's tool. Many old sewers, however, are too thickly covered with foreign growths to be susceptible to any scarifying method other than the sandblast, and sufficient compressor capacity should be provided to convert the gun into a sand blower where necessary.

*Engineer of Sewer Design, St. Louis, Mo.

The surface obtained with the scraping tools is shown in Fig. 2 and can be compared with the appearance after the untroweled gunite coat had been applied.

As for the advisability of troweling, it is fairly evident that where the sewer is already overcharged the additional capacity obtained by a smooth surface is worth the cost, even though it is necessary to apply a rather wet flush coat before floating. It is estimated that the natural gunite finish will justify a coefficient of 0.015, and that troweling will reduce this to 0.013, both values to include not only the head lost in friction but that lost in eddies due to incoming laterals and manhole shafts. Taking the sewer in question as an example, this two-point decrease is equivalent to a 9% increase in capacity at an additional expense of about 25c. per lin.ft. of sewer. Fig. 3 shows the rather crude results obtained with a sharp-cornered metal float. On any future work wooden floats with rounded corners will be used.

Hitherto little has been said of the added life given an old brick sewer by filling the joints with cement mortar applied under pressure. St. Louis has many miles of such sewers built before the advent of portland cement, which today are in varying stages of decay. In the great majority of cases the damage is confined to the brick joints, which are eroded to depths varying from several inches on the invert to almost nothing around the arch.

FIG. 4. OLD SEWER WITH BRICK INVERT WEARING OUT

FIG. 5. SEWER WITH BRICK MASONRY ON POINT OF COLLAPSE

In many cases the second stage has been reached where the invert bricks are being washed away as the joint erosion progresses. Fig. 4 shows a case where the entire three rings are gone and the foundation clay is being attacked. The collapse of the sewer will eventually follow, although the life of an apparently unsafe brick arch is remarkable. Fig. 5 shows a case which has been in practically this same condition for over a year.

It is proposed to treat such sewers with a mortar coating of the minimum thickness, troweling where justifiable, and where the invert has disintegrated to a dangerous extent, to flume the flow and reconstruct the invert with vitrified brick up to the point reached by the gunite. On the foregoing work both the gun and compressor are operated on the street, only the nozzleman, one signal man and the surfacers being in the sewer.

Special Treatment of Temporary Wall—The new Essex power station of the Public Service Electric Co., Newark, N. J., is designed for future extension on the land face. The red-brick buildings are closed off here by temporary walls of galvanized corrugated sheet. These have been painted red, and by special selection of colors in several trials they have been made so close a match to the brickwork that at a short distance they appear part of the permanent work.

Aids to Traverse Computations

By Horace Andrews*

Engineering News of May 6, 1915, on page 896, published a query as to a demand for traverse tables for each 10″ of arc. In a reply which was published in abstract Aug. 5, 1915, the writer of this article referred to certain books of tables. As many requests have been received for further details regarding these books, the following fuller notes may be of interest.

Probably the most recent and accurate table of natural sines and natural cosines is that of Prof. W. Jordan (1842-1899), called "Opus Palatinum," after Rheticus. It was issued in a second and corrected edition in 1913. The first edition appeared in 1897. The table gives sines and cosines to seven places of decimals for each 10″ of arc throughout the quadrant. This work, of 270 pages, is published by the Hahnsche Buchhandlung, Hanover and Leipzig, 1913, price 7 marks. A review, in November, 1897, stated that Professor Jordan had in preparation a more extended table, giving all six functions—*sin, cos, tan, cot, sec* and *cosec*—but his death on Apr. 17, 1899, seems to have prevented or delayed its publication.

A similar book to that of Jordan, entitled "Tables Containing the Natural Sines and Cosines to 7 Decimal Figures of All Angles Between 0° and 90° to Every 10″, Etc., by C. L. H. Max Jurisch, Examiner of Diagrams, Cape Colony, Capetown, Michaelis and Braun" (printed by Breitkopf and Härtel, Leipzig), is said to have appeared in 1884, and has been used to some extent in Germany. Sossna publishes, in 1901, a list of errata of this work and points out that these are with one or two exceptions likewise found in the tables of Rheticus and Pitiscus, which were evidently used as a basis by Max Jurisch. Sossna states, in 1896, that he himself had computed a six-place table of natural sines and natural cosines for each 10″, and that he had used as a basis of comparison the 7-place log tables of Dr. C. Bremiker, all discrepancies revealed being invariably traced to errors in the ancient tables of Rheticus (1514-1576) and Pitiscus (1561-1613).

These ancient tables were to 10″ and 10 places of decimals and 10″ and 15 places of decimals respectively. They were published in 1596 and 1613 (see "Encyclopedia Britannica," 11th ed., Vol. 26, p. 328). Despite the errors of these tables—estimated by Sossna as 1,000 in number—it is stated in the encyclopedia article referred

*125 Lancaster St. Albany, N. Y.

to that "they remain to this day the ultimate authority and also formed the data from which Vlacq calculated his logarithms."

The monumental work of Rheticus, which kept computers constantly at work for twelve years, was soon superseded by logarithmic tables. The extending use of computing machines has again revived the use of the natural circular functions, after 300 yr. of dormancy. Confidence in the accuracy of the tables of Jordan is increased by the knowledge of correspondence between Sossna and Jordan, whereby all known errors were eliminated. The few discovered during the 15 yr. of use of Jordan's tables have been corrected in the 1913 edition.

Traverse tables, giving multiples of natural sines and cosines, are well exemplified in "Tafeln zur Berechnung rechtwinkliger Coordinaten im Auftrage des Herrn Finanzministers bearbeitet von C. F. Defert," etc. (second improved edition; Berlin, 1874; Verlag von Julius Springer, Monbijouplatz No. 3, xxx + 97 pp. 9x11½ in.). This gives multiples 10, 20 . . . 90 times *sin* and *cos* for each 1′ of arc, with subtables for decimal parts from 0.00 to 0.99, also tables to be pasted separately on cardboard and used if angles subdividing the minute of arc are employed. For a distance of 1,000 this table gives results to three places of decimals.

Defert's table is, in the opinion of the writer, about of the same degree of convenience and of more agreeable typography than the following, which is equally precise: "Traverse Tables Computed to 4 Places of Decimals for Every Minute of Angle Up to 100 of Distance," etc., by Richard Lloyd Gurden (London, Charles Griffin & Co., Stationers' Hall Court, 1880; 270 pp., 9¼x14 in.). Angles given to seconds of arc are less easily employed with Gurden's tables than with Defert's.

It is evident that a traverse table giving multiples of *sin* and *cos* for each 10″ of arc would be extremely large and inconvenient, while Jordan's *sin* and *cos* tables are primarily designed to be employed with computing machines. With such extensive surveys as those of the United States Geological Survey, which is said to run from 1,000 to 2,500 mi. of traverse lines a year, a computing machine might profitably be employed, but many surveyors will find such infrequent use for multiplying machines that they will prefer tables of products.

There are three of these known to the writer. The earliest is "A. L. Crelle's Rechentafeln" of 1857 (500 pp., 8½x12 in.), which passed through many stereotyped editions, all of which neglected to give multiples of 10, 20 . . . 90, and thus interrupted the use of the tables in an annoying manner. This defect has been rectified by the new edition of 1908, by O. Seeliger. The table is a very valuable help to an engineer for many purposes and gives products of any two numbers under 1,000. Its price is 15 marks.

"Ludwig Zimmerman's Rechentafeln," in its "Grosse Ausgabe" (Liebenwerda, 1896, 205 pp., 7½x10 in.; 5 marks), gives products of any figure of two places by any of 4. Its typography and paper are excellent and its price extremely low. It offers, on its title-page, a sum of 10 marks for any error discovered.

A more recent, extensive and costly work is that of Dr. J. Peters (Berlin, George Reimer, Lützowstrasse, 107; 500 pp., 9x14½ in.; 15 marks; postage 2.80 marks). This work, having the same scope as Zimmerman's, would be preferred by most computers, even at the higher price.

For computing traverses where natural sines and cosines to four places of decimals would suffice, as in all stadia work and in many other cases, the tables of Zimmerman and Peters give excellent results, though the work of a computing machine is more rapid if, as stated to the writer in reference to a well-known American machine, "the new Comptometer permits striking as many digits at once as required, and makes multiplication so fast that the product of 0.9187 × 916.83 is completed easily in 5 sec., far less time than would be required merely to find the page in the table for 0.9187."

The little four-place table of O. Seiffert (Braunschweig, Friedrich Vieweg & Sohn; 33 p., 5x9¼ in.) is to be recommended, as it gives not only natural sines and cosines but also a table of $\sqrt{2}\ sin$ and $\sqrt{2}\ cos$ for the checking of results. A duplicate computation for all traverse work is of course most expedient and in Prussia is required by government regulations.

A more convenient, simple and rapid method of traverse computation was developed by Prof. E. Hammer in "Project for a New Method of Computing the Projections of Traverse Lines" (*Zeitschrift für Vermessungwesen*, June 11, 1908; 16 pp.). Professor Hammer proposed tables giving natural sines and cosines for each 10′ or 20′ of arc, multiplied by numbers from 1 to 250 consecutively (under the supposition of metrical measurements and 250 meters as a maximum length of line), to be supplemented by a graphical table or a slide-rule for the neglected minutes and seconds of arc and the metrical subdivisions to centimeters, or even to millimeters, if thought advisable. Hammer's project has recently been brought to fulfillment by Dr. Artur Grünert in his "Tafeln zur Berechnung der Koordinaten von Polygon und Kleinpunkten" (Stuttgart, Verlag von Konrad Wittwer, 1914; vii + 181 pp.; 7½x10¾ in.; 8.50 marks.) For 10-min. intervals all products of sin and cos up to 300 are contained in this table. The graphical table for subdivisions is, commendably, supplanted by auxiliary tables whereby a double interpolation is secured in a simple manner. In the prospectus of this work the example is given: Sought 81.75 m. × sin 10° 04′ 20″.

From p. 39 of tables, 81 × sin 10° 0′ =	14.07
From the proportional-part table on the same page, for 75 cm. and tabular difference of 17 (14.24 — 14.07)	0.13
From auxiliary table, for 4′ 20″	0.10
81.75 × 10° 04′ 20″	14.30

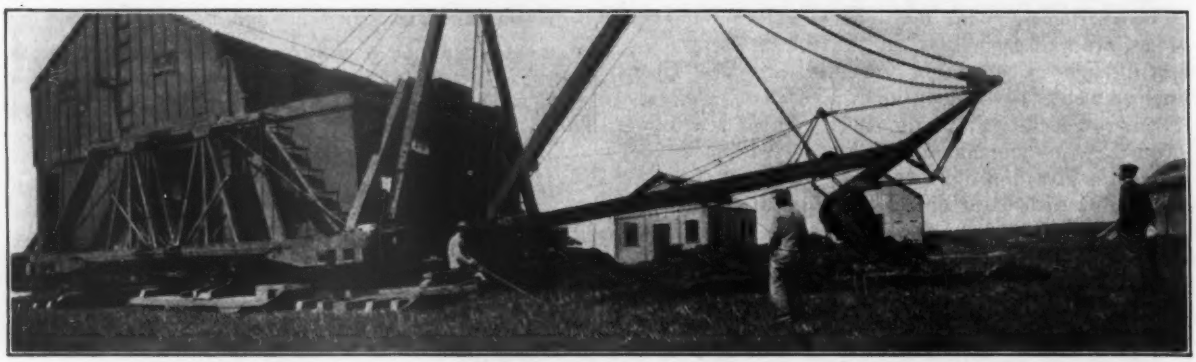
Grünert's table is carefully reviewed by K. Ludemann (*Z. für Vermess.*, July 21, 1914) and its use contrasted with Gauss' "Polygonometrische Tafeln," which in Prussia enjoys general esteem and popularity, with the result that Grünert's results are at least as exact as those of Gauss and require only two-thirds the time. The scope of Grünert's tables, 300 m., exceeds Hammer's suggestion of 250 m., yet for use with the foot as a unit of measure they would seem to fail. A similar table for foot measures might well be founded on Grünert's and based on 2-ft. intervals, or less conveniently 3-ft. intervals, up to 1,000 of distance, without expanding the work immoderately. Such a table would be a notable advance upon existing means of traverse computation.

German books not out of print may be ordered from their publishers at the prices quoted, which should be sent by international money order, with an addition of 20% to cover cost of postage. Some of the tables—Crelle's and Peters'—can be had with descriptive matter in English or French.

Dredge Walks Along Over Trench

A "walking dredge" excavated trench on a section of the new aqueduct for the water-supply of Winnipeg, Man., Canada. The dredge was carried on four sleds, one at each corner, with an intermediate sled or pad at each side. The side pads supported a post each, on which the dredge was lifted slightly by means of chains, winches and a hoisting engine. With the weight off the corner pads, the structure is moved forward by chains pulling on the legs, which normally rest on the pads. The lifting and the pulling are continuous during the operation of the dredge, which moves forward, straddling the trench. The speed is about 6 mi. per day. The barnlike structure on top of the machinery house (see view) is the workmen's bunkhouse.

This dredge was used by the J. H. Tremblay Co., contractors for the westerly 18 mi. of aqueduct. During the past season it was not worked to capacity because of the shallow trench. It is being used at present in replacing the spoil bank as backfill over the aqueduct. James H. Fuertes is consulting engineer on the project. W. G. Chace is Chief Engineer.



DREDGE "WALKING ALONG" OVER TRENCH. WORKING ON 18-MI. SECTION OF WINNIPEG AQUEDUCT

Topographic Survey of the City of Dayton, Ohio

By R. H. Randall*

In connection with the design of a comprehensive sewerage system for Dayton, Ohio, by Metcalf & Eddy, Consulting Engineers, Boston, Mass., and F. O. Eichelberger, City Engineer, the city is making a complete topographic survey.

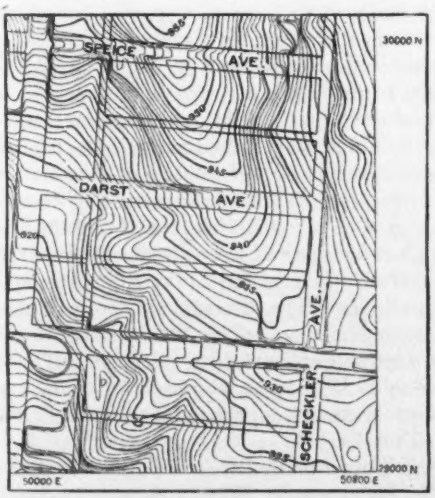
As some portions of the city had already been mapped for flood-prevention purposes at the time of the beginning of the city survey, a scale of 1 in. = 200 ft. was adopted to agree with the previous work. A contour interval of 1 ft. was selected owing to the large areas of flat or gently sloping ground, although the city has an extreme relief of about 250 ft.

The area covered by the survey is approximately 25 sq.mi., and the horizontal control is entirely by primary traverse, no triangulation being undertaken because of the prohibitive cost of such work and the small area involved.

These traverses are run in circuits, so as to form a network of lines covering the entire survey. They are systematically adjusted upon themselves, according to their closure errors, and the coördinates of all stations are computed. The instruments used in running all traverse lines were Bausch & Lomb theodolites reading to minutes, and 100-ft. steel tapes. In all, 61.9 mi. of traverse was run, the average error of all circuits being 1 ft. in 6,197 ft., and the perimeter traverse inclosing the entire net having an error of 1 ft. in 32,884 ft.

The vertical control was run with Bausch & Lomb semi-precise levels and target rods reading to 0.001 ft. In establishing 42 permanent bench marks, 48.5 mi. of levels was run, the average closing error being 0.009 ft. per mile. The entire level net was adjusted by the United States Geological Survey method, based on the closing error and length of circuit. All bench marks are of the standard types used by that organization.

The transit-stadia method of taking topography was tried at the beginning of the topographic work, but soon proved unsuited to work demanding such great detail and accuracy. Plane-tables were then purchased, and the resulting topography has been cheaper in cost and greatly superior in quality to that taken by transit.



PART OF DAYTON TOPOGRAPHIC SHEET
(REDUCED ONE-HALF)

Before going into the field each plane-table sheet has plotted upon it the exact sheet edges and all traverse stations occurring within its area. All plotting is done by

*Engineer in charge of topographic survey.

rectangular coördinates, the origin of coördinates being placed sufficiently southwest of the survey to insure all signs of coördinates being plus. Each map sheet is 27x20.5 in. in size, and is made of the best-quality double-mounted drawing paper, the grain of one of the doubled sheets being at right angles to the other.

Practically all work in open or residential districts is done by compass traverse running between the plotted control points. The horizontal closing error was arbitrarily fixed as 7 ft. on the mapping scale, but both compass and force- and backsight lines generally close with errors not exceeding 4 ft., and these are sometimes caused by the unavoidable changes in the paper due to different degrees of humidity. No limit was set for the vertical closing errors, but in all except very hilly districts these seldom exceed 0.2 ft., and the average is 0.1 ft.

The number of readings per acre averages 9, and ranges from 3 to 30 in various types of topography, the latter number being in quarries, gravel pits, etc.

Two plane-table parties are in the field, each composed of a topographer, recorder and rodman, and from 20 to 90 acres per party is covered in a day. To date the largest week's work for one party has been 0.34 sq. mi.

After the sheet is completed in the field it is returned to the office and inked in three colors—black being used for culture, brown for contours and blue for drainage. All street and alley lines are checked with the city's plat books before inking.

Reproductions in one color—black—are being made by photolithography.

Following are some cost data:

Primary Traverse	Per Mi.
Field work	$10.08
Office (computing and adjusting)	5.40
Total	$15.48
Semiprecise Levels	
Field work	$11.39
Office (computing and adjusting)	2.99
Total	$14.38
Plane-Table Topography	**Per Sq.Mi.**
Field work (based upon field time and expenses of 14 sheets)	$288.18
Inking	90.00

Report on Concrete Poles for Electric Railways

The Committee on Power Distribution of the American Electric Railway Association (A. S. Richey, Worcester, Mass., chairman) in its recent report for the San Francisco convention has summarized its tests and experience of several years in the following eight paragraphs:

1. Failure of a pole is always due to stretching of the reinforcing rods on the tension side.

2. A failure is always preceded by the appearance of hair-line cracks in the concrete on the tension side, at rather frequent and regular intervals from the ground line up.

3. It is advantageous to use a high grade of reinforcing steel to secure the maximum tensile strength.

4. Plain round reinforcing rods are essentially as satisfactory as twisted or other rough rods, because in general the rods will elongate before they slip in the concrete.

5. A larger number of small rods is preferable to a smaller number of large rods, as a better distributed reinforcement may be secured for a given amount of steel and a greater bonding contact surface is presented to the concrete.

6. The reinforcement need not be uniform throughout the length of the pole, but may be stepped off as the top of the pole is approached.

7. A pole with uniform reinforcement will break at the ground line, while one with tapered reinforcement will break at some point above the ground, depending on the taper of the reinforcement.

8. A concrete pole has an element of safety in it, as a failure of the pole will not in general allow it to fall to the ground. It is difficult at times to pull over a pole after failure, even though it is inclined at a large angle from the vertical.

The committee presented complete specifications for manufacture of reinforced-concrete poles. The following paragraphs are of interest:

Weights—Variations in weights of poles shall not exceed 5% over or under the weights specified in the order.

Reinforcement—The area of reinforcing rods shall constitute practically 3% of the cross-sectional area) of the pole at the ground line, and shall be made up of 16 reinforcing bars placed as shown. The modulus of elasticity of the reinforcing steel shall be at least 30,000,000.

Pole Attachments—Holes shall be provided in the pole for steps, cross-arms, span-wire attachments, bracket arms and other fixtures.

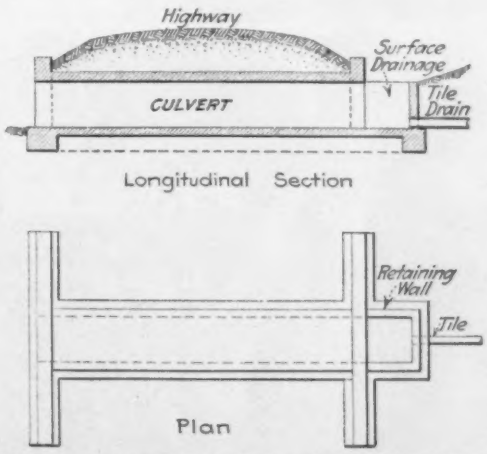
Test—One per cent of the number of poles (minimum of one pole) may be tested to destruction, and 10% of the total number of poles may be tested for deflection with the specified loading, all at the expense of the contractor. Any additional testing that may be required by the company shall be at its expense. The poles to be tested may be selected by the company.

Rejection—The failure of 40% of the poles tested to meet the requirements as herein specified shall be sufficient cause for rejection of the entire order.

Culvert with Ditch Connection

By F. O. Nelson*

The use of reinforced concrete for highway culverts has facilitated the design of a convenient method of retaining the earth at the upper side of the road where (as is frequently the case) the culvert is at the head of a drainage ditch or where it is desirable to fill the ditch above



CONCRETE HIGHWAY CULVERT WITH BASIN FOR DRAINAGE DITCH CONNECTION

this side of the road. The arrangement shown in the accompanying drawing has been found very satisfactory. It serves the interests of the highway and the adjacent landowners, and may be modified to suit individual cases.

The drawing shows a box culvert having wings parallel to the roadway. When the wings are at an angle of about 45° or more with the roadway a straight wall between the outer ends of the wings may be used. A floor for the culvert and for the basin formed by the retaining wall is of course essential to prevent scouring by the water. The area of the basin inclosed by the retaining walls is made equal to the cross-sectional area of the barrel of the cul-

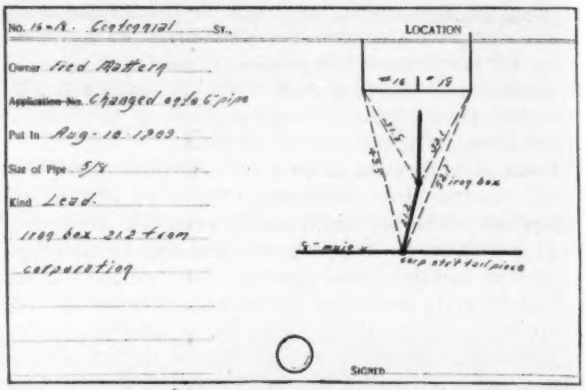
*Drainage Engineer, Toledo, Iowa.

vert. This is ample, but probably not extravagant, for small culverts when the low cost and the reduced chances of being obstructed by drift are considered, though the capacity of the vertical opening is much greater than that of the horizontal opening of equal area.

For tile outlet protection this plan, at very low cost, eliminates the washing around the ends of the bulkhead wall and the undermining, both of which give trouble on steep grades.

NOTES

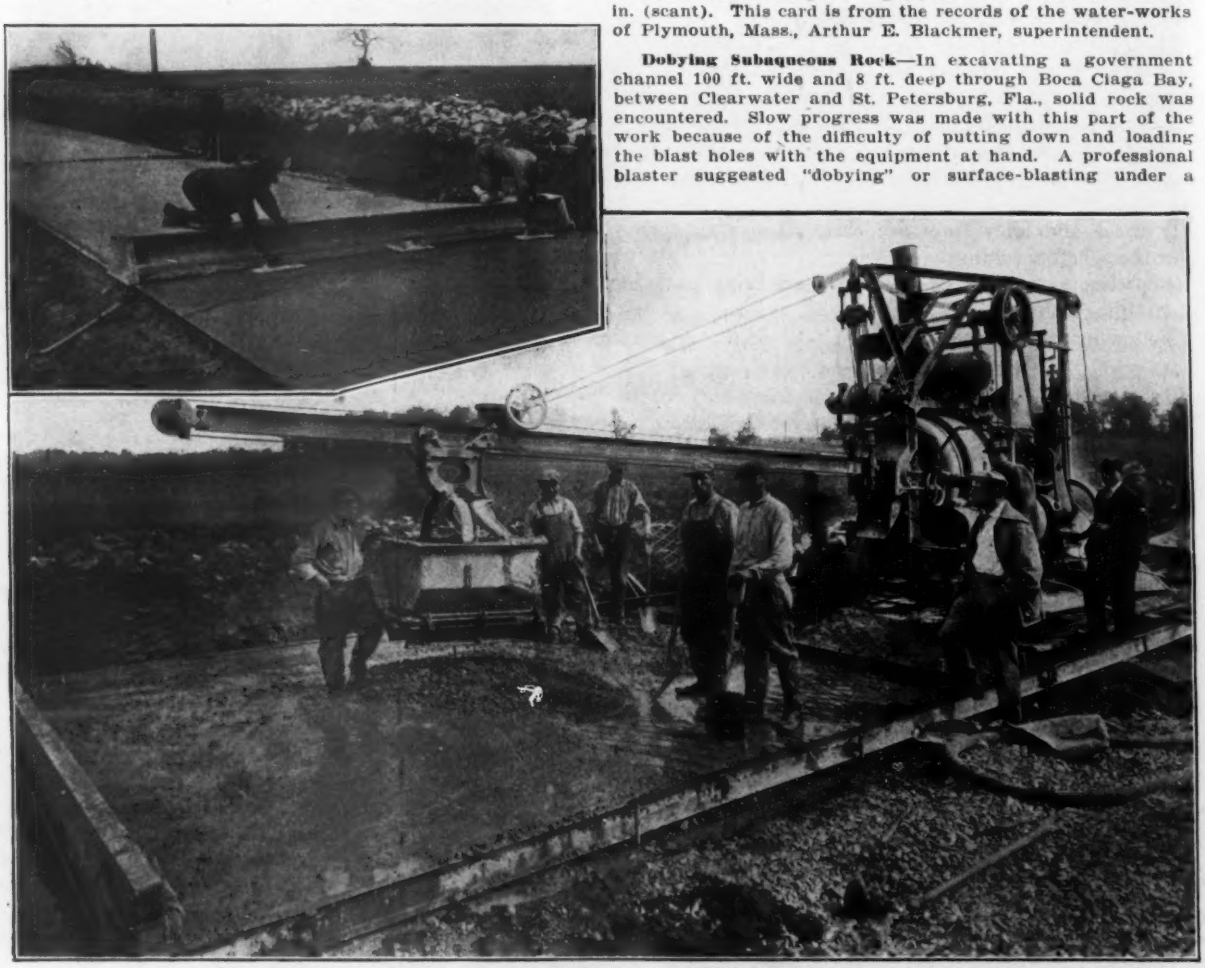
The First Concrete-Paved State Highway in Pennsylvania is now under construction between Easton and Bethlehem. The pavement is 16 ft. wide, 6 in. thick at the edges and 8 in. at the center, and there is a crushed-stone shoulder on each side. The pavement is reinforced with triangular mesh wire



PLYMOUTH, MASS., WATER-WORKS SERVICE RECORD; ABOUT ONE-HALF ACTUAL SIZE

the attention of "Engineering News." The actual size is 4x6 in. (scant). This card is from the records of the water-works of Plymouth, Mass., Arthur E. Blackmer, superintendent.

Dobying Subaqueous Rock—In excavating a government channel 100 ft. wide and 8 ft. deep through Boca Ciaga Bay, between Clearwater and St. Petersburg, Fla., solid rock was encountered. Slow progress was made with this part of the work because of the difficulty of putting down and loading the blast holes with the equipment at hand. A professional blaster suggested "dobying" or surface-blasting under a



CONSTRUCTION OF THE PENNSYLVANIA STATE HIGHWAY BETWEEN EASTON AND BETHLEHEM

and is built in 39½-ft. sections separated by tar-paper expansion joints. The crew and outfit shown in the accompanying views have laid from seven to eight sections (280 to 320 ft.) per 8-hr. day. The work is being done under the direction of the Pennsylvania State Highway Department, which has never had very much money to spend on new construction. The cement was contributed by various cement companies.

Water-Works Service Records—Keeping records of service connections in a card file is quite general in water-works practice, but there is much diversity as regards the size and make-up of the card record. The card shown in the accompanying illustration is one of the neatest which has come to

tamping cap; and instead of using a mud cap the blaster suggested that the water over the rock would serve. The scheme was tried with success. The rock was a soft coral.—"Du Pont Magazine," November, 1915.

Cracks in Walls of Cage-Construction Buildings are ordinarily thought to be impossible. A Pittsburgh building, however, shows a pronounced crack in its rear wall for the full height of the lower story. Local engineers have attributed it to racking effects of wind; but if this were true, the crack should be diagonal or multiple, whereas it is straight and nearly vertical. Whether expansion effects could produce such a crack is a matter of dispute.

Editorials

Recovering the Losses of Defeat

Despite the overwhelming defeat of the revised constitution in New York State on Nov. 2, there is still ample room for improvement through legislative enactment in a number of matters which the new constitution was expected to mend. The engineers who worked together—and separately—to improve the constitution in ways of particular concern to them might well direct their efforts for a few years to come to making the most of the present constitution and to devising individual amendments for submission to the legislature and subsequently to popular vote. Just what direction these efforts should take could best be determined after further study—perhaps by committees or a joint committee of the engineering societies centering in the state. There is good reason to believe that besides study of the sort indicated much educational work should be done to bring home to the people at large such anomalous conditions as the choice of a State Engineer by popular vote, the existence of a medley of 150 overlapping and conflicting departments, commissions and bureaus, issuing 50-yr. bonds to pay for improvements with a life of 5 to 10 yr., and changing the organization and personnel of executive departments charged with the construction and maintenance of great public improvements with every turn of the political weather vane. The remedy of some of these and of many other evils is dependent on neither constitutional amendments nor legislative statutes, but rests simply on sound and strong public opinion. Every engineer in New York State should do his part to make public opinion in these matters sounder and stronger.

⁂

Is There a Reaction Against Municipal Ownership?

The rejection of municipal purchase of street railways at Cleveland and Detroit on Nov. 2 is taken in some quarters as a further indication of the same reactionary tendency as was exhibited in the defeat of the proposed new New York constitution on the same date and various constitutional amendments in New Jersey and California during the preceding two weeks. Neither street-railway vote, however, can be taken as a good reading of the municipal-ownership barometer, for local disputes clouded the issues in each city.

In Detroit the city administration, the street-railway commission and political leaders generally favored the purchase. Strange though it sounds, the municipal ownership league opposed it—probably through failure of the ordinance to name a price. The labor vote was against it. The railway company was willing, but apparently not anxious, to sell. In Cleveland the municipal-ownership ordinance created no great activity. It was initiated by the local Socialists, and neither of the leading candidates for mayor openly favored it.

These and other fall elections, as they affected utility companies, showed discordant manifestations that would puzzle the most astute politician. Toledo turned down an ordinance, giving the electric-railway system a 25-yr. franchise, so that in the Ohio city the existing chaos is continued. In Cleveland two suburban electric roads were granted rights for improvements, one of them a subway under the business section, and a new union railway-station project was approved. In Pennsylvania a constitutional amendment was passed allowing Philadelphia to raise its borrowing capacity from 7 to 10% of the assessed valuation, the increase applying only to money for self-sustaining improvements. This makes effective the $6,000,000 appropriation ordinances already passed for the new Philadelphia rapid-transit plans. It is locally expected that the new mayor-elect is more or less hostile to the present (reform) administration's program for the extension of the city's rapid-transit facilities.

⁂

Making the Elevated Railway a Thing of Beauty

For some time past the rapid transit field has been dominated by subways. They have a grip on the popular fancy that threatens to bar all future elevated-railway construction. It is true that Philadelphia is going ahead with a rapid-transit program that includes long stretches of elevated railway. But elsewhere the subway is the only type of line considered. Thus, a comprehensive report on Detroit transit, made a few months ago by well-known consulting engineers, discussed street cars and subways exhaustively but ignored elevated railways; even the word "elevated" did not occur in the whole of the bulky report. So also in Cleveland all rapid-transit ideas are based on subways. Broadly, the same is true wherever rapid transit is thought of.

The elevated railway is cheap and efficient. But it has fairly earned the popular dislike. It is noisy and it darkens the street; also, many persons say it is ugly. The noise is the chief objection. Only those who have lived and worked alongside the metallic roar can realize the condition personally, but anyone can gage the effects by noting the depreciated state of property and population within several hundred feet to either side of an elevated railway line.

The ugliness of the elevated-railway structure is a question of taste. Probably no one ever lost a dollar, or ever had his comfort or convenience reduced, by the æsthetic shortcomings of an elevated railway.

Yet the essential motive in the design of the Queens Boulevard elevated structure, described in this issue, seems to have been a purely æsthetic one—to produce an ornamental structure. At the same time a structure of greatly reduced noisiness has been built—having solid floor, massive framework, well-cushioned roadbed, and parapet walls that will confine the spread of noise from the car wheels. It is a fair question whether the adoption of the design was not dependent just as much—or more—on noiselessness as on beauty.

It may be conceded that a very handsome, not to say monumental, structure has been built on Queens Boulevard. The structure is a most valuable object-lesson as to the possibilities in the way of elevated-railway construction. It furnishes a living proof that skill and study can produce a satisfactory elevated structure. Under these circumstances, the subway should not have the exclusive right-of-way in future consideration of rapid transit.

Progress in Foundation Work

Founding a heavy structure on soft bottom has ever been the test of the builder's skill. It is the same today, although the modern builder has the help of a great array of methods and machines for dealing with water, penetrating treacherous ground and forming a solid structure to distribute the superstructure loads. Independent thought and initiative are vital to success in foundation work—today as much as two centuries ago.

The share of the constructor as distinguished from the designer, in this exercise of originality is worth special attention. Reference to the construction of foundations for torpedo-boat berths at Charleston, on another page, will show an interesting example of the demands for original planning on the contractor's part. In that case a novel design was to be built, but standard or ordinary designs are only less likely to require the contractor to devise new methods. This is true of all construction fields, but nowhere more than in the foundation field. Here the integrity and strength of the finished structure depend in large measure on the builder's skill in producing conditions that will realize the various assumptions underlying the design.

Foundation construction is at the same time one of the most difficult and one of the most promising fields of the engineer's activity. The engineering of foundations is by no means as far advanced as other fields of design, on the score of our ignorance of the materials—the soil—if for no other reason. There is ample opportunity for progress, therefore, in the matter of theory and design. But the opportunities for new work on the constructor's side are even greater.

How Electrifying a Railroad Increases Capacity

The substitution of electric traction for steam in the Philadelphia terminal of the Pennsylvania R.R. and out on one of the important lines, as described elsewhere in this issue, is a significant railway development of the moment. One of the early statements in the article is that the change was made to increase the capacity of the Broad St. Station and to relieve congestion. It has become a generally accepted fact that the substitution of electricity for steam does give an appreciable increase in the traffic capacity of a railway terminal or line, but probably few persons not directly engaged in electrification work realize the ways in which this is secured. Comparison of the Pennsylvania's Philadelphia terminal under the old and the new operation demonstrates unusually well by what actions this improvement is secured. The increase in capacity is here effected, for one reason, because motor-car trains replace those hauled by locomotives and shifting of empty trains from one track to another is eliminated. There is also an appreciable gain in capacity due to the more rapid acceleration and to the shorter length of tracks occupied by a string of motor cars than by a train with steam locomotive.

The 8% increase in terminal capacity, as a result of the electrification of one line alone, is notable. The ultimate increase in station capacity when all six suburban lines are electrified will be equally notable, although it is possible that the total increase for the terminal will be slightly under the 40 to 50% which might be expected.

There are many points connected with this installation which are of more than passing interest. For instance, the assurance of the railroad that the savings in operating cost over steam will at least pay the interest on the additional investment will be an eye-opener to many railroad engineers who still fight shy of electric traction because of the additional investment required. Also, the selection here of a high-tension alternating-current system gives food for reflection—bearing in mind that the Pennsylvania's New York terminal, less than a hundred miles away, uses direct current at 600 volts. This alone is a good indication of the progress in alternating-current traction that has been made in recent years. Had the single-phase system been advanced as far as now, it is probable that the direct-current system might not have been employed at New York—remembering the probabilities of an eventual trunk-line service between these cities and beyond.

Buying a Pig in a Poke

The old doctrine of *caveat emptor* is still good law. Let the buyer beware! Let him specify precisely what he wants, and then see that he gets it. This procedure guards him not so much against bad faith—for as a rule there is not a great deal of bad faith in business—but against misunderstandings. The second party cannot know precisely what the first party expects to get unless the qualities desired are so described that they can be measured. What is "a good gray-iron casting?" It depends on the purpose; and if we leave it to the foundryman to judge what kind of casting will suit our purpose we are bound to accept the result of his guess, right or wrong.

The following letter from a reader of *Engineering News* tells the story of a case where the buyer did not specify, but depended on a vague type name:

Sir—Some time ago a concern in which the writer is interested placed an order with a steel foundry for the following:

Six tray lips, pattern A-1044. . . . These lips are to be of vanadium cast steel 0.35 to 0.45 carbon, annealed. . . . Care should be taken that the metal in the cutting edges is solid and of the best quality. . . .

These castings were shipped direct to a coal dock where they were to be used. Some of them broke within a few hours, and all broke within a comparatively short time after being put in service, and did not give as much service as some ordinary openhearth steel tray lips which had previously been used. Payment of the bills was refused. The steel foundry brought suit for payment, in which the following testimony was developed:

The superintendent of the steel foundry testified that 6½ lb. vanadium (alloy about one-third pure) was placed in each pot of 1,600 lb. steel, which would amount to from 0.15% to 0.16% (calculation shows 0.1354%); that vanadium put into castings varies from 0.05% to 0.15%, sometimes higher if it is ordered higher; also that the amount of ferrovanadium lost in the melt varies a good deal with the temperature of the steel, from nothing up to one-half of one per cent. A chemist stated that he found the vanadium content of the castings to be 0.03% and 0.04%; carbon content, 0.375% and 0.384%. A metallurgist stated that the addition of 6½ lb. of ferrovanadium (33⅓% vanadium) to 1,600 lb. of steel would

be equivalent in round numbers to 0.14% vanadium, and that there would be at least 20%, and possibly 30%, of that lost. The result would be about 0.10% vanadium remaining in the steel. He stated further that there is a recognized "standard vanadium steel," not less than 0.15% of vanadium, and that castings containing only 0.03% and 0.04% would not be castings of "standard vanadium steel," and that the two castings in question showed numerous gas cavities and blowholes. Three other witnesses testified that after the castings were broken they observed numerous holes.

Notwithstanding the foregoing testimony as to the vanadium content and defects of the castings, a decision was given for the plaintiff.

The question now comes up, which interests all purchasers of steel castings of special alloy or otherwise: Is there a standard vanadium-steel casting, and to what extent must full and complete specifications, chemical as well as physical, be given in a simple order when not covered by a term contract in which these elements are fixed? Let the steel foundries come forward and declare themselves. Would all of them, under the circumstances, claim that these were merchantable castings?

Probably no alloy name made up of the names of chemical elements is a proprietary designation, whether nickel steel, carbon steel, manganese steel or vanadium steel. Any alloy of nickel and steel may be called nickel steel. Where practice limits the range of composition it can hardly ever define the composition so closely that detailed specification is unnecessary.

Even simpler names, made up of commonly used words whose meaning any law court might be presumed to know, are made the subject of laboriously developed specifications. "Sound yellow-pine trestle timber" is an example. Experts in timber have thought it worth while to spend years of labor in getting up a satisfactory specification for this material. It is not sufficient that the meaning of each word, "sound," "trestle," etc., is known and can be established before any court or jury.

The activities of the American Society for Testing Materials are a convincing demonstration of the need for specifying the qualities of material. In that society the two materials whose production is most highly standardized—steel rails and portland cement—have given rise to the most active fighting over specifications. And those who did the fighting were not men lacking knowledge of what the trade practices are, but they were manufacturers and buyers of these materials. They recognized that it is impossible to rely on a mere type name or on a fixed trade practice.

⁂

The Philadelphia Engineers' Club has mobilized and has started an invasion of Pittsburgh. The *Gazette Times* of the latter city headlines a news item as follows: "Philadelphia Engineers' Club Is Starting Campaign with 1,000,000 Goal." In a four-day expedition the club expects to get 1,000 new members, says the item. At that rate, working 360 days in the year, the club would reach its goal in 11 years and 38 days, due allowance being made for the number of members it already has, without deduction for resignations or deaths. This is exactly the same time that an express train would require to reach the sun, if it started from the earth at a speed of 1,000 mi. per hr. Other analogies are that neither the express train nor the club will reach the goal in question, and also that both the express train and the Philadelphia Engineers' Club are interesting tools of civilization.

Letters to the Editor

Railway-Track Maintenance

Sir—Your editorial in regard to defective railway-track maintenance is timely. There should be some sort of legislation to relieve conditions on some of our railways. The writer is not an advocate of anti-railway laws or regulations, many of which are very inadvisable and contribute nothing to the comfort or safety of the traveling public; but there is a real need for outside regulation of railway-track maintenance, which fact I believe will be admitted by most railway officials themselves.

What are the causes of poor spiking, loose bolts, low joints, wide gage, bad alignment, poor drainage, soft track, bad bridge approaches, weak bridge members and similar examples of defectively maintained track? It is nothing more or less than the lack of track labor to give the proper constant attention which a good-riding railroad track demands. As long as railway management pursues the policy of cutting the maintenance forces to the lowest possible point every time a financial retrenchment is decided upon such bad track conditions will prevail.

Of course it is to be admitted that maintenance is the great "regulatory reservoir" for railway-operating expenses, but if we consider the present practice there seems to be no defined point below which it is dangerous as well as uneconomical to proceed.

The procedure of some of our railroads is to spend money lavishly at times in renewing ties and rail, ballasting track, etc., and then suddenly reduce track forces to the limit, thus losing the effect of the good work that has been done.

The writer has known one railroad west of the Mississippi to reduce track forces on 6-mi. sections to one laborer, besides the foreman, and this in the middle of the summer. The same ratio of reduction applied to the bridge gangs.

Not considering the matter of comfortable riding and risk of injury or death to the public, is such a policy economical to the railroads? It is probable that the expense incident to derailment and wrecks, increased cost of restoring the track to shape after such a "lean" period, loss of traffic due to unsatisfactory freight delivery or uncomfortable traveling conditions will be greater than the saving in maintenance expense.

Derailments at such a time increase to such an extent that engineering parties are kept constantly busy making surveys to determine their causes. The real cause is evident to their chief, but the remedy is unfortunately not in his power to prescribe. It has been suggested that if the section men were organized into labor unions, as are some of their fellow railway employees, such reductions in forces might be made inadvisable.

The writer would propose as a remedy that the railroads be required to maintain a minimum number of laborers on track- and bridge-maintenance work. This number could be figured on the track-mile as a basis, of course making allowance for the number of main and side tracks, or it may be that the train-mile might be a better criterion. Such regulation could be enforced by our state utility commissions and the Interstate Commerce Commission if the proper authority be extended to them. Is not such legislation more pertinent to the needs of the traveler than "full-crew" bills or even two-cent fare laws?

It is certain that it would have, in secret, the unqualified approval of most superintendents and engineers of maintenance, not to mention the roadmasters and supervisors.

G. F. SCHLESINGER,
Assistant Professor of Civil Engineering,
Ohio State University.

Columbus, Ohio, Oct. 25, 1915.

British Engineer Submits an Activated-Sludge Query

Sir—In reading the many interesting articles published in American technical journals concerning the activated-sludge process of sewage treatment I have failed to grasp, or to quite appreciate, the lines upon which American engineers are pursuing their investigations. Little appears to be said about the ideal results which experimenters desire to obtain. To realize what I mean, let us start at the bedrock of the methods used at present. The process has been explained to the writer by stripping the theory of all scientific verbiage and putting it in its naked simplicity.

First of all, in modern plants, sewage is usually treated in a percolating filter, by spreading it evenly over the surface and allowing it to pass through the material to the main underdrainage system. The liquid, in passing through the filter, flows over the surface of the medium, and the more vesicular the character of the latter, the better.

On starting up a new filter the sewage shows but little change after passing through, but as time goes on, a growth shows itself in the body of the filter on what may be termed the stationary framework, and colonies of bacteria accumulate there, which attack the sewage and effect purification or oxidization of the organic matter. These bacteria, being mainly aërobic, an ample supply of air is necessary in order that they may thrive—hence the necessity for adequate ventilation of the underdrainage. That, in brief, is the theory of the percolating filter as it appears to an engineer. Let us take a given volume of the stationary framework from a ripe filter and carefully remove the coating or growth throughout the material. We shall then have a certain volume of what may be termed sludge rich in aërobic bacteria, which may be referred to as the activated sludge of this particular process.

In the new process the active sludge is circulated throughout the sewage in the presence of air, as against the present-day practice of passing sewage in thin *films* over active sludge retained on a stationary framework, as in the case of percolators, for example. The real problem, then, is as follows: To ascertain the most economical method of applying air to sewage with maintenance of complete circulation of the activated sludge, without any formation of dead banks of material. It may be futile, as well as unnecessary, to attempt complete oxidation of a liquid which contains a considerable percentage of grit and garbage. Nevertheless, a plant, no matter how well designed, may fail in its object or become thrown out of gear if the liquid received for treatment by this process is not as near "grit free" as is reasonably practicable. I would therefore suggest to experimenters that more regard be paid to this important point.

Having an adequate volume of grit-free sewage, the next step appears to be the accumulation of activated sludge. To obtain this commodity it is necessary, I believe, to construct a tank or vessel and to pass air under pressure into the body of the liquid in such a way that the suspended particles or solids are constantly in suspension and distributed uniformly throughout the body of the liquid. To bring this about, or rather to produce the activated sludge, may take several weeks of continuous blowing, but once obtained in sufficient quantity we have all we require for experimenting further. The problem may now be said to resolve itself into one of reproducing in a tank the changes which take place in a percolating filter. We have our tank, which represents the walls and floor of the filter. We have our colonies of bacteria and the air necessary to support their life, and all we require to do is to provide means for supporting an even distribution of this life throughout the body of the tank, other than by allowing the bacteria to adhere to a stationary framework such as stone, slate or clinker material.

The engineer has to solve this problem in the most economical manner, and the question is how should it be effected. Compressed air will both insure sufficient agitation and effect the desired result as regards preserving uniform contact and even distribution of the sludge in the liquid. But at what cost?

I will leave the matter there. Perhaps American engineers may consider whether my remarks are worthy of further comment as a lead in the direction of solving the activated-sludge problem and its practical application.

OSWALD J. WILKINSON,
Consulting Engineer to the Manchester Corporation Rivers Committee for Sewage Disposal.

Manchester, England, Oct. 20, 1915.

NOTES AND QUERIES

Is the Jitney Responsible?—Commenting on the suggestion that the good pavements of Los Angeles and the high speeds which good pavements always encourage may explain the large number of automobile accidents in Los Angeles (page 757, "Engineering News." Oct. 14, 1915), a correspondent asks if the extended use of the jitney in the city named may not be the real cause of the large number of automobile accidents there. It is argued that the bus drivers are irresponsible and careless and, besides increasing traffic problems by their numbers, are inconsiderate of pedestrians' rights.

Southern California Rainfall Records—A committee of the Southern California Association of Members of the American Society of Civil Engineers is coöperating with the United States Weather Bureau in establishing rain gages in the mountainous regions of southern California. Six stations have recently been established by William Mulholland along the line of the Los Angeles Aqueduct. The committee desires to know of any records kept by engineers in this territory and seeks all possible coöperation with individuals or firms. G. S. Binkley, 514 Central Building, Los Angeles, is chairman.

Full-Load Secondary Stresses in Elevated-Railway Bents

By L. R. Manville*

SYNOPSIS—Full-load bending of the cross-girder in an elevated-railway bent produces dangerous stresses in the columns. Formula. Amount of stress in New York design. Means of reducing or eliminating the stress.

Secondary stresses have always been more or less ignored by designers in their calculations for proportioning structures. These stresses are difficult and tedious to calculate, and besides, there appears to be a general feeling of doubt as to their importance or even as to their existence. Since engineers in the past have largely neglected them and yet the structures seem to stand the test of time, the same old practices are followed today and the secondary effects passed by.

However, the fact that these structures stand up does not warrant a total disregard of secondary stresses. Very conservative unit-stresses were usually employed in the past, and the resulting large factor of safety provides for many unknown contingencies before actual failure. In structures with good details, much higher unit-stresses could have been employed in design if the secondary stresses had been considered, for one of the main factors of uncertainty would then have been eliminated. A saving of material would even have been effected in many types of structure. In a certain class of structures, however, in which the secondary stresses are abnormally large, the procedure just suggested would not have saved any material even with very high unit-stresses, but it would have produced a more intelligent design; that is, the material would have been placed to much better advantage.

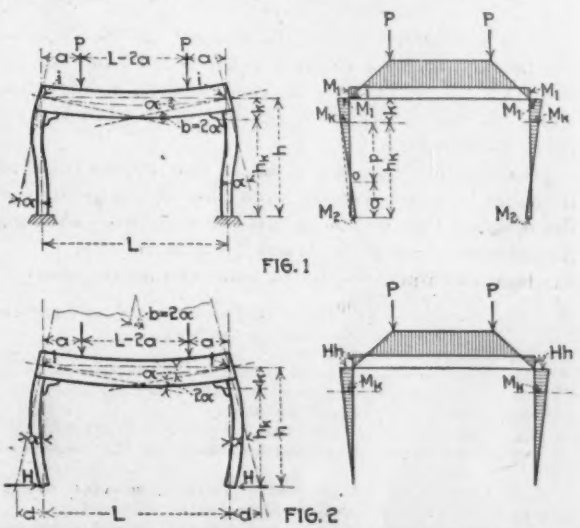
The American Railway Engineering Association has just completed a very extensive and complete investigation of the magnitude and location of secondary stresses in railway bridges. It has gone into the methods of making the calculations and has taken extensometer measurements on the various structures calculated, thus affording an excellent opportunity to verify the accuracy of secondary-stress computations. The report of this work (Vol. 15) demonstrates clearly that secondary stresses are real and that they amount to a considerable portion of the primary stresses. Thus, in structures with fairly good details, the secondary stresses amounted to 50% or more of the primary stresses.

The theory being sound and the stresses real, it is necessary that the designer of any structure of importance should have a clear understanding of the secondary-stress principles. He must also be familiar with the methods of design for reducing these stresses to a minimum; in some cases he must even go to the trouble to calculate the stresses. Few structural failures can be explained without consideration of the stresses other than primary, the latter as a rule being fairly moderate.

*Office of Engineer of Structures, Exterior Zone, New York Central R.R. East of Buffalo, Grand Central Terminal, New York City.

While in the case of trusses the calculation of secondary stresses becomes somewhat involved, yet in the case of plate-girder construction (as used in the ordinary type of elevated-railway structures) the problem is fairly simple, and the writer has worked out equations which can readily be applied by any draftsman by simply substituting easily computed factors.

In a transverse bent with two columns carrying a cross-girder, the columns being effectively connected to the girder with kneebraces, as the transverse girder deflects under vertical loads the joint at the column connection turns and a resulting bend is placed in the column. In other words, the columns produce a restraining effect upon the girder, tending to resist its downward deflection. To be sure, where the columns are long and slender their resistance to the transverse-girder deflection becomes very small; on the other hand, in the case of short stiff columns the neglect of this secondary bending would mean an error of some 15% in the girder stress. But while the bending of the column tends to reduce the girder stresses, the columns at the same time get large increase of stress due to their lateral bending. Hence the consideration of secondary stresses in such a structure may be confined to the columns, and as the development of the equations to be stated later shows, the secondary bending stress will be a maximum at the top of the column.



FIGS. 1 AND 2. DISTORTION AND MOMENT DIAGRAMS FOR ELEVATED-RAILWAY BENT

Upper sketches for fixed column-base; lower sketches for hinged column-base

Similar bending stresses in the columns, but longitudinal to the structure, would also be found in elevated-railway structures where contraction and expansion are provided for in only every third or fourth bent. The following equations, however, are not intended to cover such cases. The solution of this type of problem is much simpler than the one under consideration in this article.

By making the assumption that the point of counterflexure in the column is at a point one-half the depth of the column, the solution is made easy, and the assumption is no doubt very close to actual conditions after expansion or contraction has taken place.

Secondary Stresses under Full Load

The maximum secondary bending stress per square inch, in columns of transverse bents, occurring at the outer fibers of the columns in the plane of the bent, (at the foot of the bracket) can be expressed by a readily applied equation[1] as follows:

$$f = \frac{Pwa\,(L - a)(3k + 2h_k)}{2I_1h^2k + 2LI_2\,(4k + 2h_k)}$$

In the foregoing equation practically no approximations have been made, and it is believed the solution is exact with all practical conditions considered and provided for. A slight approximation is made in assuming a constant moment of inertia for the transverse girder.

A special feature of considerable importance should be noted in connection with the equation—the effect of the bracket upon the bending curve of the column. The column is considered to remain straight in the depth of the bracket, or for a distance k downward from the top, which is no doubt true in most modern construction, owing to the comparative rigidity of the bracket and column connection.

While at first sight the equation may appear involved, it should be noted from an inspection of the meaning of the notation that all the factors are very easily obtained. The notation (see Figs. 1 and 2) is as follows:

- I_1 = Moment of inertia of transverse girder about its neutral axis;
- I_2 = Moment of inertia of column about an axis perpendicular to the plane of the bent;
- E = Modulus of elasticity;
- M_1 = Moment at the intersection of neutral axis of transverse girder with that of column (point i);
- M_2 = Moment at foot of column;
- M_k = Moment at lowest effective point of bracket;
- P = Any symmetrical vertical load on the transverse girder;
- h = Height of bent to neutral axis of transverse girder;
- h_k = Height to lowest effective point of bracket;
- k = Portion of column above bracket (considered rigid; that is, this portion remains a straight line after flexure takes place);
- a = Distance of load P from vertical axis of column;
- L = Width of bent, c. to c. columns;
- a = Angle between vertical and axis of column at point i after distortion;
- w = Extreme width of column in plane of bent;
- f = Maximum unit secondary bending stress in the column;
- H = Horizontal reaction at the foot of the column;
- o = Point of counterflexure in column.

General Assumptions—Point of intersection i of the neutral axes of the column and girder remains fixed as to location, but free to turn when vertical loads are applied to the girder.

The right angle between column and girder remains unaltered after flexure.

Moment of inertia of both column and girder is constant. In the equations, the moment of inertia of the columns is only required for the portion below the bracket, where the moment of inertia is constant.

In the foregoing equation the base of the columns is considered restrained. This condition of absolute fixity of the base of columns in most cases of design does not obtain; the inequalities of the masonry work and the "give" in the anchor bolts, etc., tend to relieve the bending at the column top to some extent. A slight "give" in the base would materially relieve the maximum bending at the bracket, the point where the equation just stated furnishes a value for the maximum bending stress in the column. Another extreme condition would be to figure the column base as hinged. With these two conditions before him, the engineer can judge from the details of the bent in question about what bending stresses should be provided for.

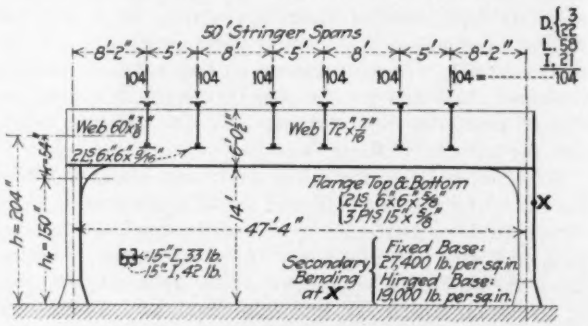


FIG. 3. SECONDARY STRESSES IN NEW YORK ELEVATED BENT

Bent of new city elevated lines. Dimensions and loads taken from article in "Engineering News," May 20, 1915

For the condition of *hinged base*, the maximum unit-stress in the column (at the foot of the bracket) would be

$$f = \frac{Hh_kw}{2I_2}, \text{ where } H = \frac{Ph}{I_1}\left[\frac{a\,(L - a)}{\frac{2h^2k}{3I_2} + \frac{h^2L}{I_1}}\right]$$

Applying these equations to a typical bent such as that used in the design of the New York new rapid-transit lines, the bending stress in the column at the foot of the bracket will run as high as 27,000 lb. per sq.in. for the condition of fixed column-base, and 19,000 lb. per sq.in. for hinged base. Even a mean of these two extremes, or 23,000 lb., is a surprisingly high stress. Under the direct loads considered in this computation, the axial stress in the columns is 9,700 lb. per sq.in., which with the mean of the bending stresses, makes a total of 32,700.

In Fig. 3 is given a typical elevated transverse-bent section, assuming 50-ft. longitudinal spans. Loads and all data are taken from information furnished in the article in *Engineering News*, May 20, 1915, entitled, "Design of Steel Elevated Railways, New York Rapid Transit System."

Summary and Conclusions

A study of the equations affords an opportunity to point out some pertinent facts useful in a design of this type of construction.

The equations show that long and shallow cross-girders, and short stocky columns should be avoided; in the plane

[1] This equation is derived by equating deflection angles at the point of intersection i, Figs. 1 and 2, of column and girder, taking advantage of the fact that the top of the column rotates through the same angle as the end of the transverse girder. All the equations showing the various relations between the unknowns are based upon the two fundamental equations from "Mechanics":

$$\frac{dy}{dx} = \int\frac{Mdx}{EI} = \text{tangent of the angle of deflection}$$

$$y = \int\frac{Mxdx}{EI} = \text{deflection}$$

The integration is done by taking the areas of the moment diagrams. To find a relation between M_1 and M_k the location of the point of counterflexure has to be determined. It should be noted that by assuming the flexural curve of the column straight for a distance k, the value of M_1 is considerably increased and it becomes an important factor in determining the amount of the transverse girder rotation about the point I.

of the bent, at least, the columns should be slender. However, slender columns do not afford lateral rigidity, which makes the stiff column imperative in most cases from a practical viewpoint. In any event *the bending stresses should be figured* and provided for.

In case the stiffness of the column is such that high bending stresses result, the transverse girder should then be made as deep and stiff as consistent with economy, and if possible the span length should be reduced.

A slight bevel on the ends of the transverse girder at the column connection might be made to reduce the column-bending stresses. By making the top of the transverse girder slightly longer than the bottom, a bevel would result at the column connection such that an initial inward bend would be given the columns with no live-load on the transverse girder. Then, as the live-load causes the girder to deflect, the columns would assume their normal vertical position. Only a very small bevel would be required. In the case of the transverse girder of the New York elevated bent, a bevel of ⅛-in. in 6 ft., the depth of the girder, would greatly reduce the bending stresses. To give this bevel the top of the girder should be made ¼ in. longer than the bottom.

A slight spreading of the column bases during erection would also tend to reduce these bending stresses.

If the bending stresses together with the direct axial stresses are too severe, additional material in the columns might be the best solution. It should be borne in mind, however, that material added to the columns will not reduce essentially the bending stresses, but will reduce the direct axial stresses in proportion to the material added. This method would prove expensive and more or less inefficient.

When secondary stresses are considered, higher unit-stresses might safely be used in design, because one of the elements of uncertainty is eliminated and a reduction in the factor of safety is therefore permissible. It is suggested that unit-stresses at least 25% or even as high as 50% in excess of the ordinary designing unit-stresses be permitted.

American Steel Cars Built for Russian Railways

Probably the most interesting of the car equipment recently ordered in the United States by the Russian Government is the lot of 5,000 general-service steel gondolas being built at McKee's Rock, Penn., by the Pressed Steel Car Co., of Pittsburgh, one of which is shown in the accompanying views.

The rated capacity is 110,000 lb., but a test load of 154,000 lb. was required. The car weighs 46,000 lb. and resembles a common American type. The body is 40 ft. by 9 ft. 6¾ in. and 4 ft. 4 in. high. The center sill is continuous from end to end and consists of two $\frac{5}{16}$-in. bent plates riveted to a 4-in. rolled T at the top and to 3½x3½-in. angles at the bottom. Malleable-iron center braces are provided at the bolsters and pressed-steel braces at the cross-ties. The end sills are of ⅜-in. pressed steel, reinforced with pressed channels extending between short side sills and center sills and connected to them, and are braced from points at the rear of the buffers with pressed channel-shaped braces in such a manner that the major portion of the buffing shock will be transmitted through these braces into the center sills. The bolsters and cross-ties are pressed diaphragms reinforced with plates and angles.

The bodies have doors on both sides throughout the length, which are operated by a creeping-shaft device and rest directly on the shafts when in the closed position. The chains, which wind on drums, are only employed for lifting the doors and to hold the shaft in place under them when in the closed position, as stops are provided to engage both sides of the doors when they are dropped. The ends are made of 2¼-in. planks, reinforced all around with 2½x2½x¼-in. angles, and are secured with links at the end sill in such a manner that they can be dropped into the car when not required. The sides are made of ¼-in. steel plates in three lengths, supported by seven pressed-steel side stakes located at bolsters and cross-ties, and are reinforced at the top with 4-in. rolled-steel bulb angles extending the full length. Near the bottom the side sheets are sloped in at an angle of about 60° and have a flange turned out which rests on the end sills, bolsters and cross-ties.

The Russian type of Westinghouse air-brake equipment, with 10-in. cylinder, is being applied. It differs only in detail from the freight-car equipment used in this country. The foundation brake rigging follows standard practice in this country, but involves the use of a tension spring to insure release of the brakeshoes.

The couplings, which are of the hook-and-link type, and buffers are the Russian standard, except that coil springs have been substituted in place of the volute type. Coupler forgings are of special heat-treated steels having elastic limits of 44,000 and 56,000 lb. per sq.in.

Arch-bar trucks, with 6-ft. 3-in. wheel base follow M. C. B. construction in general. Changes are introduced because of the large-diameter wheels—41$\frac{5}{16}$ in.—and the 60-in. track gage. The wheels are of rolled steel, with flange and tread similar to the Russian standard, but differing only slightly from the M. C. B.

The completed car stands 8 ft. 11 in. from rail to top of sides and is 44 ft. 4⅛ in. long over the buffers.

STEEL GONDOLA CAR FOR RUSSIAN IMPERIAL RAILWAY COMMISSION

East River Tunnel Shields

SYNOPSIS—The Flinn-O'Rourke Co. is using shields with sliding platforms but without doors. Compact hydraulic equipment. Gravel-shooting through lining to fill voids around the shell.

Compressed-air work is just beginning in four tunnels under the East River, being built by the Flinn-O'Rourke Co. for two new subway lines of New York City. One pair of tunnels extends from Old Slip, Manhattan, to Clark St., Brooklyn; the other extends from South Ferry to Montague St. The west river-headings of the South Ferry tunnels will not be driven by shields, since they are in rock (these headings will be stopped 1900 ft. out, where rock ends). But as shield work is required in the approaches, the total number of shields is twelve. All of them were built to the design shown in the drawing herewith. The shield structure was designed by the engineering department of the Flinn-O'Rourke Co. The hydraulic equipment was designed and built by the Watson-Stillman Co., of Aldene, N. J.

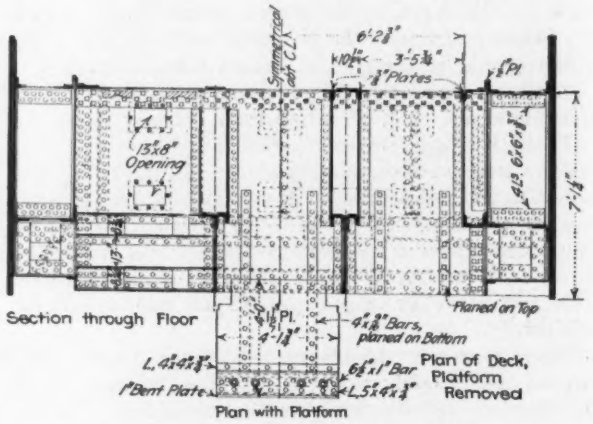
The tunnels are of moderate diameter (18 ft. and 18 ft. 6 in. respectively). There are only one horizontal and two vertical partitions, making six openings or pockets in the shield. All three partitions are of box construction; the horizontal one is deeper than the vertical partitions on account of receiving hydraulic jacks for sliding platforms.

Main Structure

The main shield ring consists of inner and outer shells spaced apart by seventeen longitudinal I-beams of 24-in. 80-lb. section. These form chambers longitudinal of the ring which contain the jacks for shoving the shield forward. The main or outer shell is a little less than 13 ft. long, of which 5 ft. 9 in. is tail. The length of inner shell, and hence of the jack chambers, is 4 ft. 3 in. The chambers are closed off at front and back by annular plate bulkheads, the back bulkhead having holes for the jack plungers. The vertical and horizontal partitions extend 32 in. forward of the front bulkhead, or within 10 in. of the cutting edge in the lower section of the shield. The front wall of the jack chambers is stiffened by 34 triangular plate-and-angle brackets connecting to the outer shell.

Hood and Sliding Platforms

A permanent hood is fixed to the front edge of the shell. The hood is sectional, to allow replacement of parts or removal in case it is preferred to work without the hood.



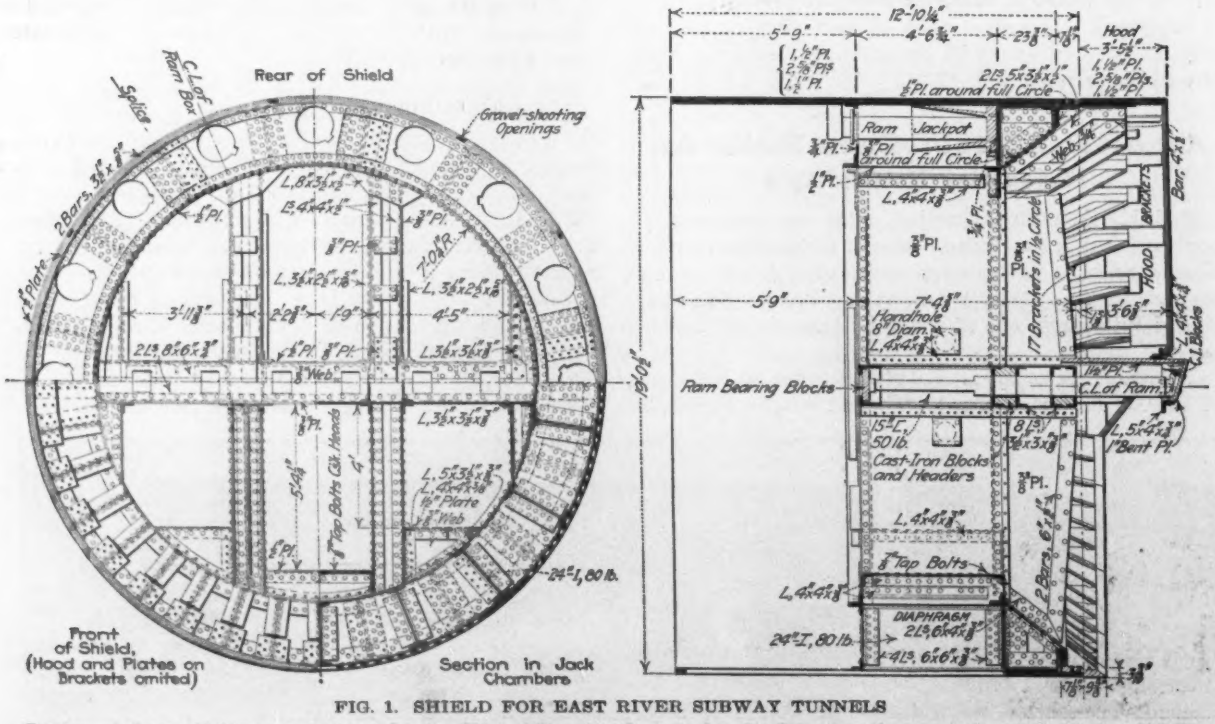


FIG. 1. SHIELD FOR EAST RIVER SUBWAY TUNNELS

Twelve of these shields were ordered by the Flinn-O'Rourke Co., six for the four tunnels now being built at the Old Slip and at South Ferry, and six for their easterly approaches

The hood segments in the upper half of the shield are of steel plate-and-angle construction, and are braced back by large kneebraces to the stiffening brackets of the front of the shield. The segments of the lower half are of cast steel.

Three sliding platforms are provided on the horizontal partition or working deck of the shield. They rest on flat-bar tracks on the upper plate of the deck, and overhang forward with an apron. Under each sliding platform are two hydraulic jacks, seated inside the deck and fastened to the apron of the platform.

The shield is not fitted with doors. The contractors do not expect that doors will be needed, nor do they believe that they would be other than an obstruction. For emergencies in which the pockets must be closed as a matter of safety, stoplog angles are riveted to the vertical partitions at their rear edge.

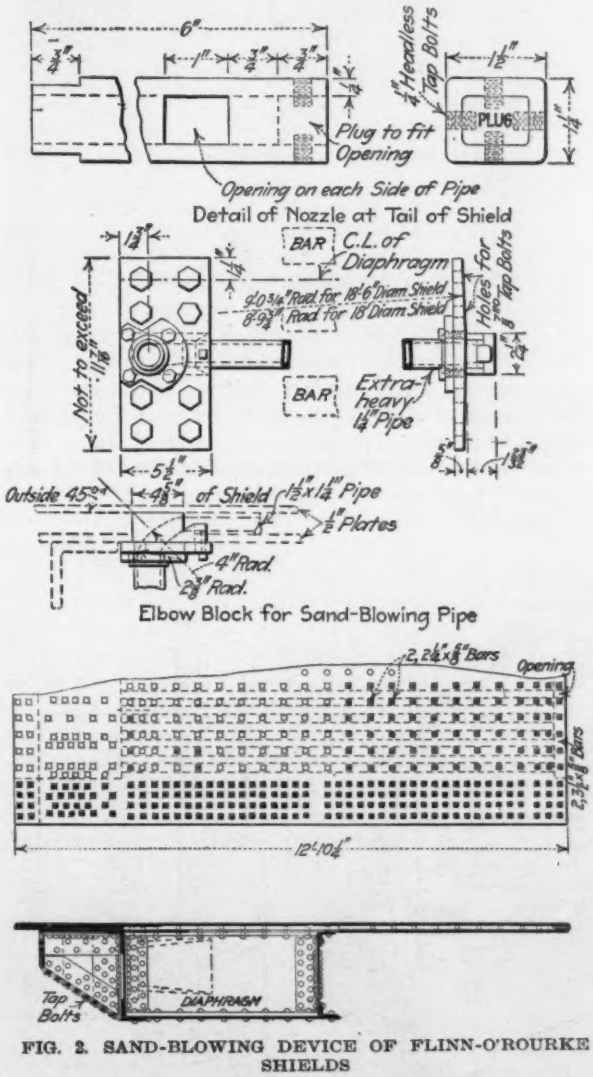


FIG. 2. SAND-BLOWING DEVICE OF FLINN-O'ROURKE SHIELDS

Sand-Shooting to Replace Grouting

A new and radical feature of the work with these shields is a substitute for grouting the void left by the tail of the shield. The scheme was originated by the contractor. As first planned, sand was to be blown out from the tail of the shield by air blast through longitudinal channels in the thickness of the outer shell. How these channels are formed is made clear by the sketch in Fig. 2 of the shield tail. The shell-plate has its outer and inner sheets separated by longitudinal bars, between which are longitudinal spaces. A certain number of these—about 12 in the whole circumference—open at the rear edge of the tail. Each of these channels has a rectangular opening to the inside, just back of the rear diaphragm of the shield, and a cast-iron angle block, as shown in Fig. 2, was set into this hole to connect with a pipe of rectangular section slipped into the rear portion of the channel and terminating just behind the rear edge of the tail. Lateral openings in the end of this pipe formed the nozzles for the sand that was blown through.

The procedure was to blow sand or fine gravel by a sandblast arrangement through these pipes while the shield was shoved forward, and thus fill the void left by the tail of the shield as fast as the void is created. The sand is dried before it goes into the sand-blast tank. By blowing air through the nozzle pipe before opening the sand valve, it was expected that all moisture would be blown out and the sand pipes dried out sufficiently to avoid clogging.

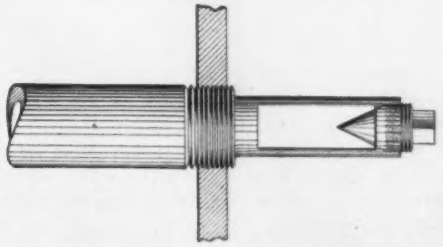


FIG. 2-A. NOZZLE FOR SHOOTING GRAVEL THROUGH GROUTING-HOLES IN CAST-IRON LINING

Within the jack chambers the channels in the tail of the shield are left open on the inside to permit insertion and removal of the sand-blowing pipes.

Gravel-Shooting Through Grout Holes

Actual trials of shooting sand or gravel through the tail of the shield did not prove satisfactory. The first attempts were made with sand. It was found that the sand if perfectly dry would blow out well, but enough moisture remained in the shield passages to cause clogging sooner or later. The idea of using sand had to be abandoned.

Gravel was then used, "birdseye" size being found best. It went in easily, and the operation was in all respects successful so long as the nozzle pipe was intact. The impact, however, wore out the nozzle pipe in a few hours, and then the passage packed full.

A new plan was then resorted to and proved a complete success—shooting in the gravel through the grouting holes in the cast-iron lining at a point just back of the shield. The nozzle sketched in Fig. 2-A is screwed into one of the regular grouting holes (one such hole is provided in every ring segment) and the tip of the nozzle with its lateral openings projects into the space left by the shield tail. A cone-shaped deflector plug at the point of the nozzle deflects the gravel out through the lateral openings. The result is to fill the void space behind the lining thoroughly well.

Some distance back of this point regular grouting with a 1:1 cement-sand grout is done, again using the grouting holes. This fills all remaining voids outside the lining and presumably transforms the gravel into a concrete.

From the relative amounts of gravel and grouting materials that are injected in these two operations it is possible to form an idea of the efficiency with which the gravel shooting is done and the amount of material saved by using gravel instead of grouting with cement. In addition to this, it must be remembered that cement grouting could not be done so close behind the shield as can the gravel shooting, without causing frequent trouble in binding the shield or in flowing forward along the shield shell. The amount of gravel injected per ring (18 in. long) averages in full working about 30 to 40 bags. About the same amount of sand and cement (added together) is used farther back for filling the remaining voids.

Hydraulic Equipment of the Shields

The equipment for shield driving, for shoving the platforms ahead, and for operating the segment erector is shown in Fig. 3. It is seen that the incoming water is led to valve blocks controlling the shield jacks and platform rams and to slide valves controlling the erector.

While 5000 lb. is available for the shield and platform jacks and 2000 lb. for the erector, only one feed line is used, this being connected to either 5000 or 2000-lb. accumulators (in the power stations on the surface), depending on whether it is desired to use the shield jacks or the other apparatus. It is, of course, not required that the erector should be worked while the shield is being driven forward.

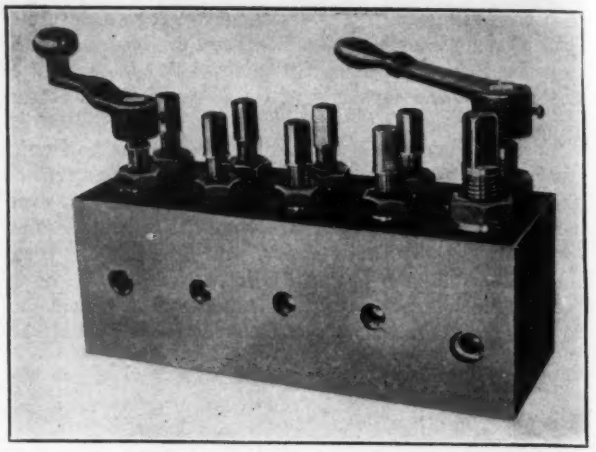


FIG. 4. VALVE BLOCK FOR SHIELD JACKS

The feed main coming down from the power station is a 1½-in. XX steel pipe with forged-steel fittings, both being stock material of the makers of the hydraulic equipment. The distribution around the shields is by 1-in. XX steel pipe with forged-steel fittings, and copper tubing from the valve blocks and slide valves. The valve blocks (Fig. 4 shows one) consist of special steel forgings with hydraulic metal bushings, seats and stems controlling the connection of each jack and ram with the pressure supply or release. The jacks can be worked singly or in groups as required to steer the shield. Close to the con-

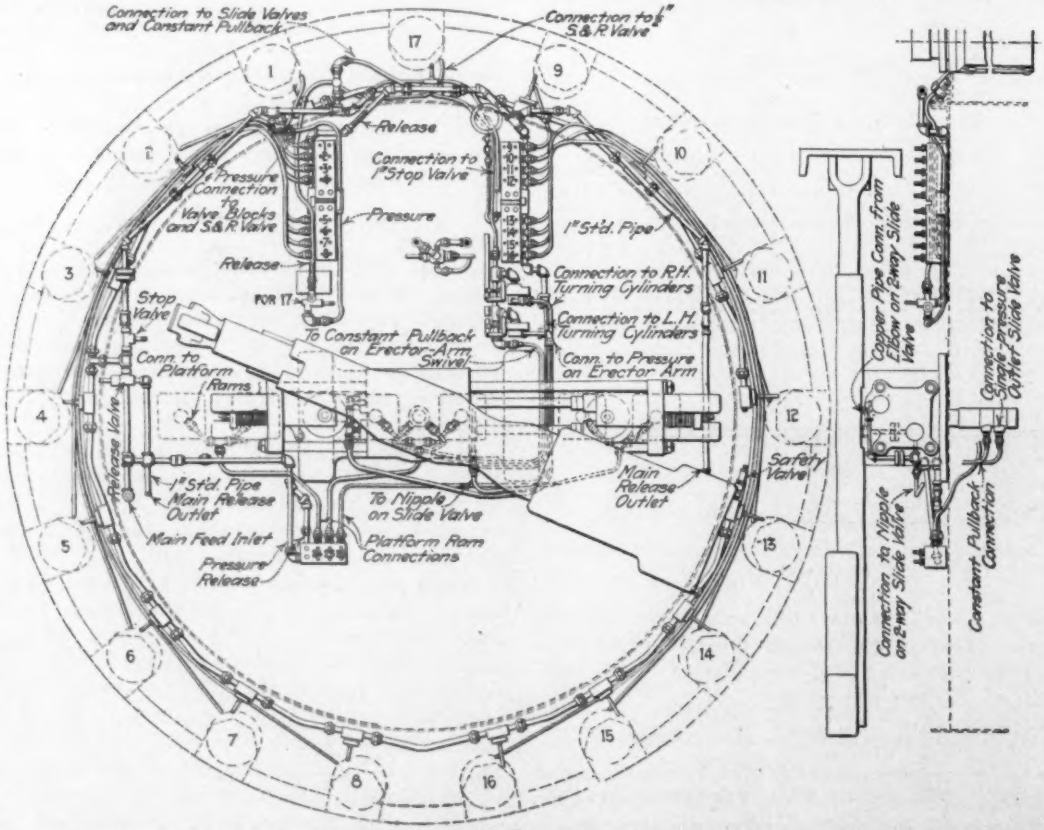

Connection to Slide Valves and Constant Pullback
Connection to 1" S&R Valve
Release
Connection to 1" Stop Valve
Pressure Connection to Valve Blocks and S&R Valve
Pressure
1" Std. Pipe
Release
FOR 17
Connection to R.H. Turning Cylinders
Connection to L.H. Turning Cylinders
Stop Valve
To Constant Pullback on Erector-Arm Swivel
Conn. to Platform Rams
Conn. to Pressure on Erector Arm
Release Valve
1" Std. Pipe Main Release Outlet
To Nipple on Slide Valve
Main Release Outlet
Safety Valve
Main Feed Inlet
Platform Ram Connections
Pressure Release
Copper Pipe Conn. from Elbow on 4-way Slide Valve
Connection to Single-Pressure Outlet Slide Valve
Connection to Nipple on 4-way Slide Valve
Constant Pullback Connection

FIG. 3. HYDRAULIC EQUIPMENT FOR EAST RIVER SHIELD JACKS, ERECTOR AND PLATFORM RAMS

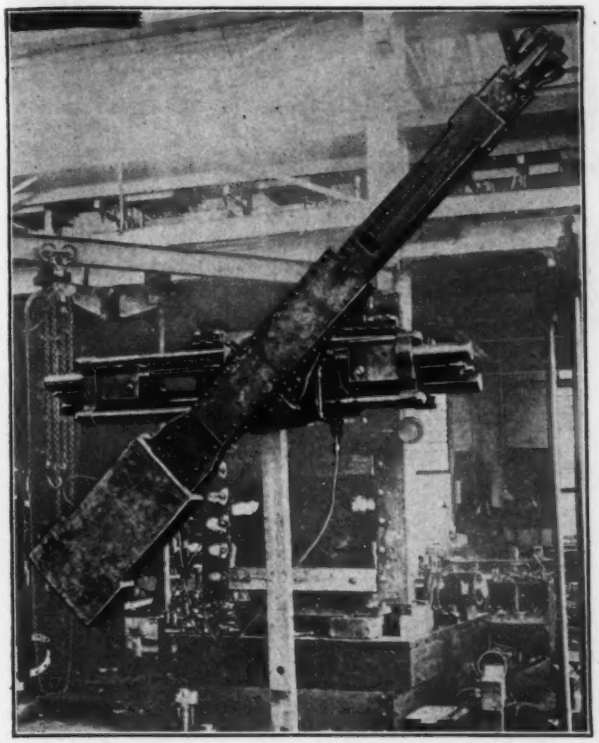


FIG. 5. ERECTOR FOR FLINN-O'ROURKE SHIELDS DESIGNED AND BUILT BY THE WATSON-STILLMAN CO.

nection of the shield piping with the main feed line a connection is taken off, through a stop valve, to the ring line to which the differential side of the shield jacks is connected for reversing.

The jacks are shown in section in the accompanying illustration. The platform rams are 3-in. differential plungers in a cylinder.

The segment erector is shown in its general position on the shield by Fig. 3, and its construction is further seen from Fig. 5. The design is novel because of the small clearances. There are four jacks, two for right-hand and two for left-hand turning. The jacks have hollow fixed rams and movable cylinders. By a specially designed arrangement of chains and sprockets, the horizontal motion of the cylinders is changed to rotary, thus

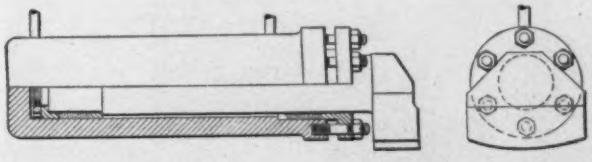


FIG. 6. SECTION THROUGH SHIELD JACK
Design of Watson-Stillman Co.

controlling the movements of the erector arm, being such as to give a little over one revolution.

The ram in the arm of the erector is of simple differential-plunger type, with pressure led to it through a swivel connection in the erector pivot. Connections for control of erector rams are shown in Fig. 3.

The tunneling work is under the management of George H. Flinn, president, and John F. O'Rourke, vice-president, of the contracting company.

Explosive Sewer-Gas Mixtures Studied at Pittsburgh

Joint studies have been made by the United States Bureau of Mines and the City of Pittsburgh, of the causes of explosions in sewers (details of which were published in the September issue of the *Journal of Industrial and Engineering Chemistry* of the American Chemical Society.) One of the largest of the Pittsburgh sewers was taken for the experiment. It is 8 ft. 3 in. wide by 8 ft. 6 in. high and 2,706 ft. long, with six manholes scattered along the length. The ordinary velocity was found by floats to be about 6.5 ft. per sec., and the depth of flow was about 8 in. The flow was 9.6 cu.ft. per sec. and the grade of the line, 0.9%. The temperature of the water was 70° F.

In the first experiment 55 gal. of gasoline (67.5° Bé.) was dumped from a barrel into the first manhole. Samples were collected at the other five manholes—at first 10 min. apart and then ½ hr. and 1 hr. One gallon of gasoline if entirely vaporized produces about 32 cu.ft. at ordinary temperature and pressure. Assuming 1.5% as the low explosive limit of vapor-air mixtures, 55 gal. would produce enough vapor to make explosive the air in 1,900 ft. of 9-ft. sewer—assuming perfect mixture. In this test the gasoline was rapidly carried past each manhole, and only the lighter portion had a chance to evaporate at any one point. The gasoline vapor was heavy, so that mixing was slow. At the second manhole, 553 ft. below the first, 0.89% of gasoline vapor was found in the air 2 min. after introduction; 2.19% was the largest quantity of vapor found in any sample. Only very small proportions were found at any manhole 10 min. after introduction of the gasoline. It was evident that one barrel poured in all at once rendered the air at any particular point explosive for only a few minutes.

In the second test the same quantity of gasoline was allowed to run in at the rate of 5 gal. per min. Some of the gas samples were taken 2 in. above the water and some 2 ft. above. The highest percentage obtained from samples taken 2 ft. above was 0.7%. Thirteen minutes after the inflow of gasoline had ceased in the second manhole, 1.14% of vapor was found 2 in. above the water; 18 min. after the inflow had ceased only a trace was left. At some of the lower manholes the gasoline vapor was not swept out so quickly; at No. 4 manhole in 16 min. the amount was 1.10%, and in 31 min., 0.12%. On ignition of the gasoline vapor at any time within 15 min. after the introduction had ceased, there would have been a rapid propagation of flame along the sewer. The test is cited as showing that 5 gal. per min. introduced continuously will produce a dangerous condition.

A third test was taken with a bulkhead built in the sewer to reproduce the condition of submerged outlet. Gasoline was dumped into the lower end (fifth manhole) at the rate of 5 gal. per min. It was found that dangerous conditions would exist with a small amount of gasoline if the outlet should be submerged by high water. The percentages of gasoline vapor in the upper part of the line gradually increased, disclosing a growing danger where the outlet was submerged for several days, while garages and dry cleaners allowed gasoline to escape.

A fourth test was made with the same conditions as in the third, except that one-half the quantity of gasoline was used (27.5 gal.). The results of analyses were in about the ratio of the quantities of gasoline used.

News of the Engineering World

Activity in Boston Harbor Work

Work has been begun on the construction of the great dry dock at South Boston, the contract for which was recently awarded by the Directors of the Port of Boston to the Holbrook, Cabot & Rollins Corporation. This work will cost approximately $3,000,000. The construction of the bulkhead and filling of flats adjacent to the dry-dock site have been completed under previous contracts.

The dry dock when completed will be the largest in the world. It is to be 1,200 ft. long and 133 ft. wide at the entrance. The foundation is to rest on bedrock and the structure is to be of concrete, with a lining of granite masonry. Plans and specifications for the pumping plant are now in course of preparation.

The United States War Department has approved the new harbor lines in South and East Boston, which include the closing of the old channel northwest of Governor's Island and filling this and adjacent territory comprising approximately 800 acres with material dredged from the harbor and channels. Since June over 300,000 cu.yd. has already been dumped there.

Contracts for dredging and placing 1,500,000 cu.yd. from certain areas adjacent to the present anchorage basin in East Boston are being prepared. It is expected that the areas to be dredged will ultimately become docking places, while the dredged material is to be used for reclaiming about 35 acres adjacent to Jeffries Point.

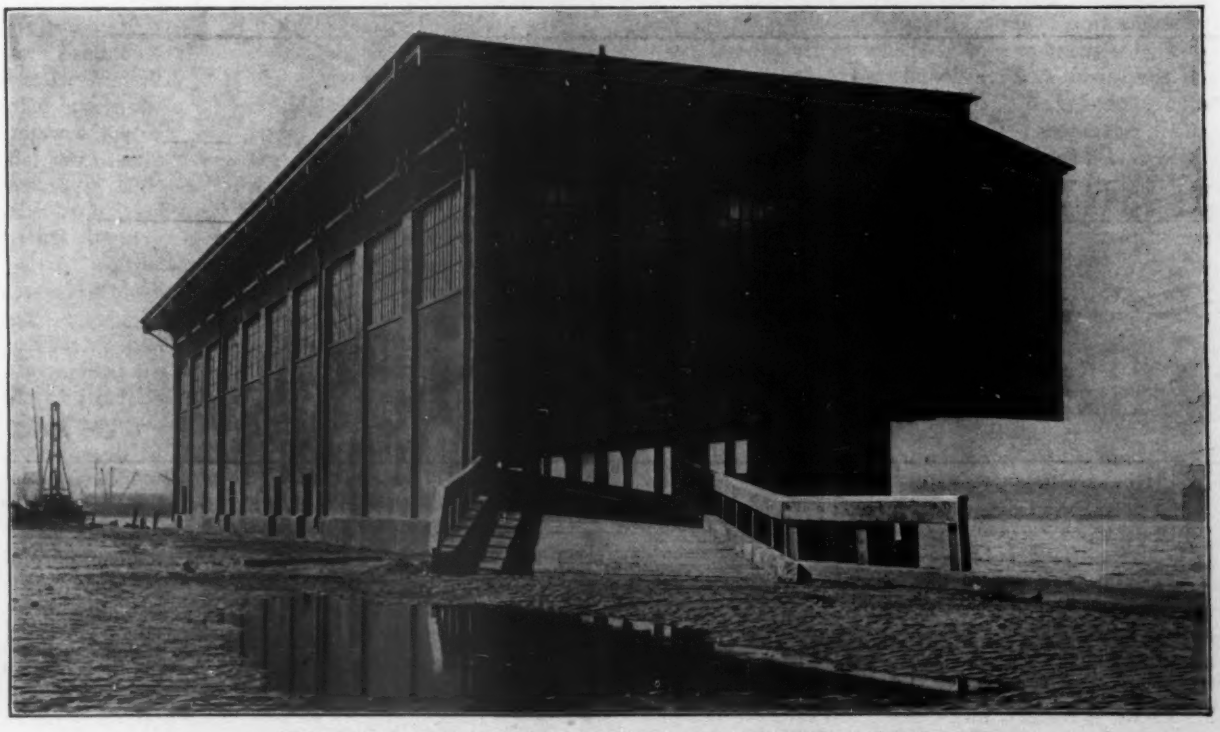
In coöperation with the Boston & Maine, Boston & Albany R.R. and New York, New Haven & Hartford R.R., an investigation is being made by the port directors to determine the volume of freight traffic between North and South Boston, which includes a zone extending from Lowell to Plymouth.

New York City Street-Cleaning Dumping Dock Completed

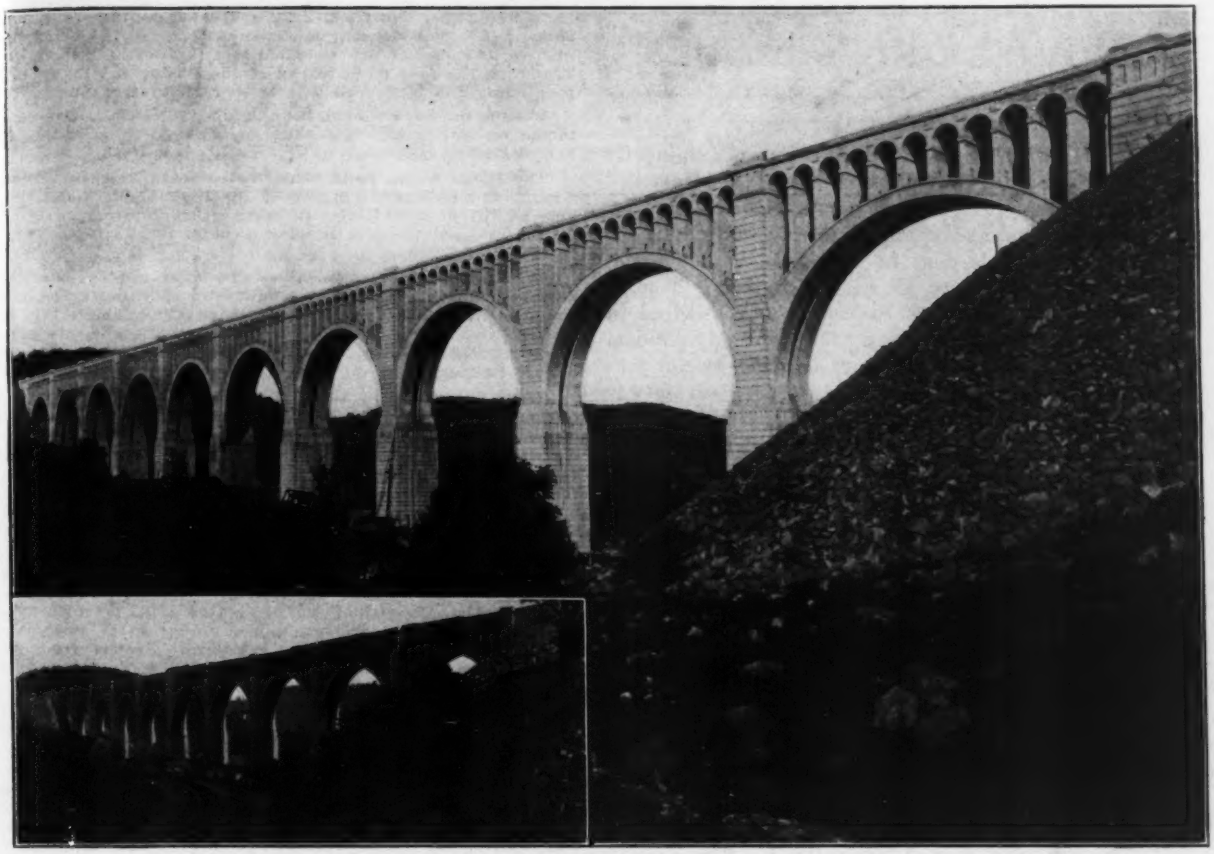
The reinforced-concrete ash-and-refuse dumping board at West 77th St. and North River, New York City, described in detail in *Engineering News* of Jan. 28, 1915, has been completed. The superstructure is steel, with stucco walls, and the roof is of ornamental steel tiles.

Refuse wagons are driven up an incline or ramp and are backed into the part of the shed overhanging the scows. The ashes and refuse fall into the scows through steel chutes, entirely eliminating the dust nuisance, so pronounced in all the old-style open dumps. The chutes are adjustable so that they can be raised and lowered to fit the height of the load on the scows. Each chute has two double-hinged doors swinging back into the chutes so as to give uninterrupted passage to the refuse. Only one door is opened at a time, depending on the position of the chute.

The structure was designed and constructed by the Department of Docks and Ferries under R. A. C. Smith,



NEW YORK CITY STREET-CLEANING DEPARTMENT DUMPING DOCK



THE TUNKHANNOCK VIADUCT AND A CURVE ON THE OLD LACKAWANNA MAIN LINE

Commissioner; R. C. Harrison, First Deputy Commissioner, and C. W. Staniford, Chief Engineer, for the Street Cleaning Department, of which John T. Fetherston is Commissioner.

Summit Cutoff Opened by the Lackawanna R.R.

On Nov. 7 regular passenger trains were sent over the new cutoff line of the Delaware, Lackawanna & Western R.R. between Clark's Summit, Penn., 7 mi. west of Scranton, and Hallstead, Penn., 14 mi. east of Binghamton, N. Y. As described in detail in *Engineering News* of Aug. 28, 1913, this is a 39.6-mi. replacement line, which saves 3.6 mi. between New York and Buffalo, reduces the maximum grade from 1.23% to 0.68% and the total curvature from 3,970° to 1,570°. All of the old 22 grade crossings have been eliminated. The new line for the most part is within sight of the old one, the radical changes noted being possible through heavy cuts and fills and by enormous viaducts. All of this construction was impossible in the early days when the original road was built. The earth excavation for the new line amounted to 5,525,000 cu.yd. and the rock 7,647,000 cu.yd. The concrete placed was 300,000 cu.yd., and 2,360 tons of steel reinforcement was used. Contracts were let in the spring of 1912. The total cost was over $12,000,000.

The most striking feature of the new line is the enormous Tunkhannock viaduct—in point of yardage the largest concrete bridge yet built (see *Engineering News* of June 13, 1912). As shown in the accompanying views, it consists of ten 80-ft. and two 100-ft. arch spans. The maximum height is 240 ft. It contains about 167,000 cu.yd. of concrete and 1,140 tons of reinforcing steel. All foundations were carried to rock.

The Summit cutoff was planned and executed under G. J. Ray, Chief Engineer of the railroad. F. L. Wheaton was engineer of construction and in immediate charge.

Large Power Project Sold

It is unofficially reported that the uncompleted hydroelectric development and aluminum works of the Southern Aluminium Co., near Whitney, N. C., have been taken over by the Aluminum Co. of America. This adds another chapter to the vicissitudes of the Whitney work (see *Engineering News* of June 11, 1914). The Southern Aluminium Co. is a French concern and on the outbreak of war stopped its work on this project. There were under construction a gravity-section concrete dam, 1,350 ft. long and 200 ft. high (maximum), a power house with five 5,200-kw. direct-current generating units (ultimately 16) and an industrial plant and town. The electrical machinery was being built by the General Electric Co., and the general contractor was the Hardaway Contracting Co., of Columbus, Ga. It is reported that the new owner will assume the old contracts and complete the entire development, with only such changes from original plans as are necessary to fit in with the Aluminum company's processes and program.

ENGINEERING SCHOOLS

Akron Municipal University

This institution, in striving to keep its work eminently practical, although of collegiate grade, has installed the coöperative system, with the students distributed for half of their time among the city's industries. In addition much work is done in connection with city departments. For instance, all the city testing is done in the university laboratories. Students recently helped in a survey of pavement conditions. The university physical director is city playground supervisor, and several students are subdirectors. A thorough housing survey was conducted jointly by the university's department of sociology, the local charity organization and the city department of health. Under the city building inspector the municipality has had a corps of competent inspectors (at no expense to the city) who received laboratory credit for their outside work.

Ambridge School of Bridge Engineering

An interesting feature of the work of the Ambridge School is a course in designing and detailing of draw-span machinery being given under F. L. Dudley. About 20 men have been enrolled in the course.

The purpose of this school is to provide an opportunity for young men in the office and shops of the American Bridge Co., Ambridge, Penn., who have not the advantage of a college education. Also, a chance for acquiring a working knowledge of the English language is extended to shop employees of foreign birth. The courses cover mathematics, mechanics, drawing, materials, bridge and building details, drawbridge design, superintendence, shop problems, office practice, English, etc. The school runs from Sept. 16 to May 31. J. E. Banks is principal.

Military Engineering Courses

It is reported that a committee from the association of Land-Grant Colleges and Universities is to appear before the Military Affairs Committees of Congress to ask the establishment of a four-year military engineering course in a number of these schools. This plan provides that each graduate from this course be taken into the United States army as a second lieutenant for one year. At the end of that year he would either be mustered out and placed in a first reserve or reënlisted for further army service.

NEWS NOTES

A 115-Ft. Construction Tower for hoisting building materials on a new structure at Second St. and Second Ave., Minneapolis, Minn., was blown over in a windstorm Nov. 1. Two workmen were seriously injured. The tower was entirely of timber and was guyed with mine wire cables.

A Fire in a Candy Factory in Williamsburg, New York City, Nov. 6, caused the death of nine employees, mostly girls. The building was a five-story brick structure with wood floors. The fire exits were found to be locked. An investigation of the State Factory Inspection Bureau is being made to determine if fire-protection laws have been properly enforced.

A 48-In. Water Main Burst Nov. 6 in Commonwealth Ave., Newton, Mass. The main is a part of the (Boston) Metropolitan Water-Works system, and its failure was probably caused by settlement and weakening after the laying of a new 60-in. main alongside. Nearly 2 hr. elapsed before the water was shut off and considerable damage was done to the boulevard and to the tracks of the Middlesex & Boston Street Ry. Traffic on the four main tracks of the Boston & Albany R.R. paralleling the boulevard was temporarily interrupted.

Ohio River Dam No. 10, near Steubenville, Ohio, was completed on Nov. 4 and the locks were placed in operation on that date.

The Barge Canal Bond Issue, Voted on in New York State Nov. 2, was carried, according to the best information available at the hour of going to press. Complete returns from all but 12 up-state counties show a majority of at least 15,000 in favor.

The Health of Street Sweepers and other employees of the Street Cleaning Department of New York City under Commissioner J. A. Fetherston will be guarded after Jan. 1, 1916, if present plans are carried out. There will be a medical director at $3,000 a year and seven medical inspectors at $1,800 a year to look after the 7,000 employees of the department.

Grade Separation at South Bend, Ind., is to be undertaken on a rather large scale, permission to require this work on the part of the railways having been granted by the state legislature. The city is to pay 25% of the cost of work at the street crossings, and is preparing to raise the necessary funds for this purpose. No plans have been prepared as yet, but the matter will be studied during the winter under the direction of Frederick J. Anderson, City Engineer, with the intention of commencing work next year.

Levee Systems Separately Provided For—At a meeting held at Memphis, Tenn., on Oct. 25, members of Congress from all the Mississippi Valley states and presidents of twenty levee boards decided to ask Congress to make the construction of a levee system a separate project from the usual river and harbor work. It is expected that $10,000,000 will be asked to complete the work. It was decided that the Congressmen should meet in Washington on Nov. 29 for the purpose of formulating a bill to be presented at the next session.

The Vote on Bond Issues at Cincinnati, Ohio, at the election of Nov. 2, resulted in the approval of one only of the four proposed issues. The $1,250,000 bond issue for parks, parkways and playgrounds, which required only a majority vote, was carried, but the other three—for a municipal auditorium ($350,000), high-pressure fire system ($400,000) and street improvements ($1,495,250)—although receiving a majority of votes, did not poll the two-thirds required by the Ohio state law. The referendum on the electric-rate ordinance resulted in the approval of the ordinance.

Sewage Treatment at Toledo, Ohio, is a problem just being taken up, partly at the instance of the Ohio State Board of Health. Both the city and Lucas County are concerned, and will probably coöperate in the investigation. The general scheme is expected to comprise a system of intercepting sewers leading to Imhoff tanks and a filter plant. The first section of the work will be for that part of the city now served by Ten-Mile Creek as an outfall sewer. The city is represented by Herbert McKechnie, City Engineer, and the county has appointed L. A. Boulay as its sanitary engineer in charge of sewage-disposal investigations.

The Chicago Street Railways are facing the problem of having to decide between the conflicting rules of two regulating bodies. The present ordinances, accepted in 1897 and administered under the direction of the Board of Supervising Engineers, forbid the use of trailer cars. The new Illinois Public Utility Commission, after an investigation of the crowded condition of the street cars during the rush hours, has issued an order to the companies to introduce trailer cars. The city authorities hold that the commission has no power to set aside ordinances in this way and the companies state that they will continue to operate according to the requirements of the ordinances as accepted by them and the city in 1897, after years of negotiation.

A Bond Issue of $2,000,000 to pay for construction work has been approved by the Sanitary District of Chicago. The engineers estimate that the completion of the widening of the Chicago River to 200 ft. will cost $1,000,000 and that another $1,000,000 will be required for intercepting sewers in Evanston and the Calumet district and for railway and highway bridges across the Calumet-Sag drainage channel. The finance committee took the position that the cost of such permanent improvements should not be borne entirely by the present generation of taxpayers and that bonds should be issued to mature between 1917 and 1935, the bonds to be for $1,000 each, with 4% interest. After the ordinance for this bond issue had been adopted it was vetoed by the president of the board of trustees, but was passed over his veto with only his own dissenting vote.

PERSONALS

Mr. Thomas A. Edison and Mr. Nikola Tesla have been awarded the Nobel prize for advance in physics.

Mr. William F. Nickel, formerly examiner in the United States Patent Office at Washington, D. C., has opened an office in the Woolworth Building, New York City, where he will practice law, specializing in the law of patents, trademarks and corporations.

Mr. James Forgie, M. Am. Soc. C. E., Consulting Engineer at 84 William St., New York City, recently a member of the firm of Jacobs & Davies, has been awarded the Telford Gold Medal by the (British) Institution of Civil Engineers for his paper, "The Laxaxalpam Aqueduct Tunnels, Mexico."

Mr. Thornton W. Price, formerly designing engineer with the American Bridge Co., and for the past two years in charge of a large part of the work on the Panama Canal coaling stations, has opened offices in the Woolworth Building, New York City, for the practice of structural and mechanical engineering.

Mr. R. L. Baldwin, Assoc. Am. Inst. E. E., for the past two years Appraisal Engineer of the State Utility Commission of Missouri, has become associated with the firm of Burns & McDonnell, Consulting Engineers, Kansas City, Mo. He will have charge of the firm's electrical work and appraisal of electrical and other utilities.

Mr. Chester F. Gailer, Assoc. M. Am. Soc. C. E., formerly Assistant Chief Engineer of the United Railways & Electric Co., of Baltimore, Md., is now associated with the Atlantic Welding Corporation, 30 Church St., New York City, as Chief Engineer. Mr. Gailer will direct the manufacture and installation of the welded rail joints manufactured by this concern and of which he was the original inventor.

Mr. Ralph H. Stearns, Assoc. M. Am. Soc. C. E., of Boston, Mass., formerly Designing Engineer of the Board of Water Commissioners of Hartford, Conn., on that city's Nepaug water-supply project, has been appointed Assistant Engineer of the Water-Supply Board of Providence, R. I. Previous to his connection with the Hartford work Mr. Stearns had considerable to do with the construction of the East Boston and Washington St. tunnels, Boston, and the Catskill Aqueduct work of the Board of Water-Supply of New York City.

Mr. Paul M. Lincoln, Past President of the American Institute of Electrical Engineers, who for many years has been prominently connected with the engineering department of the Westinghouse Electric and Manufacturing Co., has resigned his position with the company in order to devote his time to the manufacture of a maximum-demand electric meter which he has recently invented. Mr. Lincoln was graduated from Ohio State University in 1892. A biography was published in "Engineering News" of Feb. 25, 1915.

The Westinghouse Electric and Manufacturing Co. announces the following changes in its supply department: Messrs. S. A. Chase, formerly Special Representative; T. J. Pace, formerly in charge of the illuminating and rectifier divisions, and Carl G. Schluederberg, M. Am. Inst. E. E., formerly head of switchboard sales, have been appointed Assistants to J. J. Gibson, M. Am. Inst. E. E., Manager of the department. C. E. Stephens, Engineer of Lighting, has been appointed Manager of the illuminating section to succeed Mr. Pace. C. Streamer, formerly head of the order division, succeeds Mr. Schluederberg as Manager of the switchboard section, and A. P. Joseph, Assoc. Am. Inst. E. E., is appointed head of the order section to succeed Mr. Streamer. M. C. Morrow, formerly of the Philadelphia office, is appointed Manager of the appliance section, which is a combination of the former heating, fan motor and ozonizer divisions of the supply department. M. C. Rypinski, M. Am. Inst. E. E., formerly Manager of the detail and supply division of the New York office, becomes Manager of the meter section. W. R. Garret has been made Section Manager of Contracts; J. G. Worker, of stokers, and H. D. Storer, of auxiliaries, in the power department.

OBITUARY

Henry H. W. Sigourney, who practiced civil engineering until his retirement from business some years ago, died at his home in Milton, Mass., on Nov. 3, after a brief illness following a shock, at 82 years of age.

Eben R. Dyer, for the past 50 yr. Superintendent of the Portland Water Co. and the Portland (Me.) Water District, died on Nov. 8. Mr. Dyer was for many years a member of the New England Water-Works Association.

Col. T. Waln-Morgan Draper, Consulting Engineer, of New York City, the builder of the Atlantic & Danville Ry., died in that city on Nov. 8, at 60 years of age. He was a graduate of the Royal Polytechnicum in Munich and the Royal School of Mines in Freiberg, Germany. At the Centennial Exposition, Col. Draper was in charge of the mineralogical collection, after which he spent several years in the West, where he did a considerable amount of mining engineering work. In 1882 he returned to practice his profession in New York City.

Gen. George Miller Sternberg, U. S. A., retired, for a decade Surgeon-General of the United States Army and a former President of the American Public Health Association, died at his home in Washington, D. C., on Nov. 3. He was born in Otsego County, New York, and received his medical degree at the College of Physicians and Surgeons in New York City. During the Civil War and later General Sternberg did much to prevent the spread of cholera and yellow fever. At the outbreak of the Spanish-American War he organized the "typhoid-fever board" and in 1900 the Yellow Fever Commission.

Nathan S. Beardslee, a civil engineer well known in the western part of New York State through his railroad-construction work, died suddenly at his home in Warsaw, N. Y., on Nov. 1, from heart disease. He was born in New Berlin, N. Y., in 1848, and in 1869 joined the engineering corps engaged in constructing the New York, Oswego & Midland Ry. Later he had charge of the laying of the double-track line of the Erie R. R., and did a large amount of other railway location work in New York and in Illinois. He was president of the village of Warsaw for eight years, during which time the water-works system for the village was built, largely through his efforts.

ENGINEERING SOCIETIES

COMING MEETINGS

THIRD ANNUAL PENNSYLVANIA WELFARE AND EFFICIENCY CONFERENCE.
Nov. 16-19. Conference in Philadelphia.

NATIONAL MUNICIPAL LEAGUE.
Nov. 17-19. Annual convention, Dayton, Ohio. Secretary, Clinton Rogers Woodruff, 705 North American Building, Philadelphia.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
Nov. 18-19. Annual meeting at Zanesville, Ohio. Secretary, F. E. Sanborn, Columbus, Ohio.

AMERICAN INSTITUTE OF ARCHITECTS.
Dec. 1-3. Annual convention in Washington, D. C. Executive Secretary, E. C. Kempe, The Octagon, Washington.

AMERICAN ASSOCIATION OF ENGINEERS.
Dec. 10 and 11. Convention in Chicago. Secretary, Arthur Kneisel, 29 South La Salle St., Chicago.

ASSOCIATION OF AMERICAN PORTLAND CEMENT MANUFACTURERS.
Dec. 13-15. Annual meeting in New York. Secretary, Percy H. Wilson, Bellevue Court Bldg., Philadelphia, Penn.

SECOND PAN-AMERICAN SCIENTIFIC CONGRESS.
Dec. 27-Jan. 8. Washington, D. C.

Montana Institute of Municipal Engineers—The annual meeting will be held in Billings, Mont., Jan. 17-19, 1916. The president is C. E. Durland, of Billings, and the secretary is Carl C. Widener, of Bozeman.

National Lime Manufacturers' Association—The annual meeting will be held in Cleveland, Ohio, the first week in February, 1916. Fred K. Irvine, 537 South Dearborn St., Chicago, is secretary.

National Municipal League—The twenty-first annual meeting of the National Municipal League at Dayton, Ohio, Nov. 17-19, will be preceded by the second annual meeting of the City Managers' Association, which lasts three days. The first session of the N. M. L. will be held Wednesday morning, Nov. 17. Sessions of the Ohio Civil Service Association and the Ohio Municipal League will be held simultaneously. Besides the various committee reports, election of officers, etc., one session of the National Municipal League will be held jointly with the City Managers' Association. The subject for discussion will be "Has the City Manager Plan of Government Changed the Concept of City Life?" The conventions will be held at the Hotel Miami. The secretary of the National Municipal League is Clinton Rogers Woodruff, Philadelphia, Penn.

Appliances and Materials

New "Dry" Storage Battery

A new design of "dry" electric storage cell is being put on the market by J. P. Mentzer & Co., Chicago, Ill., the size being the same as the familiar commercial cylindrical "dry battery." The construction is unique. The positive and negative elements are corrugated lead strips, with the active material pasted on, rolled up into cylinders and separated by thin sheets of spongy inert material. In the center is a tube for supplying water, compensating for changes in volume of electrolyte and for venting gases. The container is an improved fiber can.

The manufacturers claim discharge capacities varying from ½ amp. for 40 hr. to 3 amp. for 5 hr. The average e.m.f. is 2 volts. It is intended to displace the ordinary zinc-carbon dry cell, having six times the working capacity and costing less to recharge than the price of a new zinc-carbon unit.

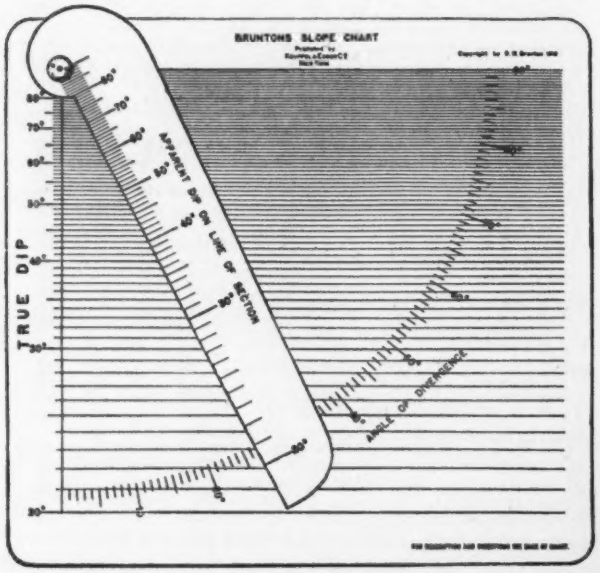
* * *

Chart Solving tan C = cos A tan B

The Brunton slope chart, recently placed on the market by the Keuffel & Esser Co., New York City, is designed to solve mechanically the equation:

$$\tan C = \cos A \tan B,$$

in which C = the apparent dip, A = the angle of divergence and B = the true dip. It is therefore of particular use in the preparation of mining and geological maps, but it may



BRUNTON SLOPE CHART FOR SOLVING EQUATION TAN C = COS A TAN B

be used for giving valley angles in the design of hoppers, ore bins, etc. Such a problem is:

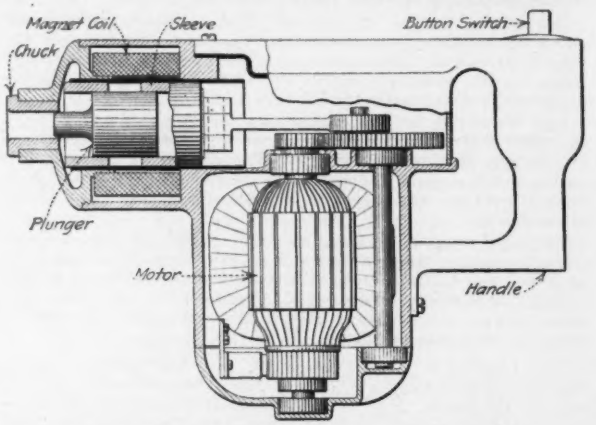
If the sides of a rectangular, hopper-bottomed ore bin have a slope of 50°, what would be the slope of the valley angles in the corners? Answer: As the bin is rectangular, the valleys have a divergence angle of 45° from the sides; consequently, if the graduated edge of the swinging arm is set at 45° divergence, opposite 50° on the true dip, read 40° on the pivoted diagonal arm.

* * *

Electric Hand Hammer and Drill

An interesting design of electric hammer and drill is now being marketed by the Western Electric Co., of Chicago, Ill. The scheme of the design is to throw a heavy plunger against the drill or chisel, there being a magnetic clutch between the motor drive and plunger to prevent transmission of the shock.

Connected to the motor through hardened gears and connecting-rod is a cylindrical sleeve having two relatively thick end sections joined together by a thin center section. This center section is as thin as is consistent with mechanical strength. The plunger which delivers the blow to the drill lies loosely within this cylinder and is not mechanically connected to any part of the mechanism. Outside of the sleeve lies the magnet coil. The coil is stationary, the sleeve moving back and forth within it. The winding is connected in series with the motor. As soon as the circuit is switched on —which is done by a button on the handle—the solenoid creates a magnetic circuit, with the thick sections of the sleeve as the poles and the plunger as the armature. As the sleeve is moved back and forth by the motor, the effect is that of moving the pole faces of the magnet back and forth. The plunger, clutched across the poles, moves with the sleeve, transferring its energy to the drill when it meets it, which it does near the end of the stroke. The plunger is slightly longer than the thin section of the sleeve, which gives it a



WESTERN ELECTRIC DRILL AND HAMMER

certain amount of over-travel and elasticity of connection. Ball bearings are used throughout.

This drill is made in three sizes for 110- or 220-volt direct current. One size weighs 14 lb. and takes 150 watts; the second weighs 25 lb. and uses 220 watts; the third weighs 50 lb. and draws 550 watts. Two sizes—150 and 240 watts—are made with universal motors for direct or alternating current. The prices range from $100 to $165.

* * *

A Double-Ended Squeegee

A new squeegee, with two pieces of split hose bolted to a metal frame, has been put on the market by the Groner-Shontz Co., Canton, Ohio. The advantage over a squeegee with only a single rubber is that when one piece of hose

GRONER-SHONTZ DOUBLE SQUEEGEE

wears out the other can be brought into use by merely changing the handle. When both are worn out new pieces of hose may be substituted by taking the frame apart.